TRB

TRANSPORTATION RESEARCH RECORD

Journal of the Transportation Research Board, No. 2298

Traffic Control Devices, Visibility, and Highway–Rail Grade Crossings 2012

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

TRANSPORTATION RESEARCH RECORDS, which are published throughout the year, consist of collections of papers on specific transportation modes and subject areas. Each Record is classified according to the subscriber category or categories covered by the papers published in that volume. The views expressed in papers published in the Transportation Research Record series are those of the authors and do not necessarily reflect the views of the peer review committee(s), the Transportation Research Board, the National Research Council, or the sponsors of TRB activities. The Transportation Research Board does not endorse products or manufacturers; trade and manufacturers' names may appear in a Record paper only if they are considered essential.

PEER REVIEW OF PAPERS: All papers published in the Transportation Research Record series have been reviewed and accepted for publication through the Transportation Research Board's peer review process established according to procedures approved by the Governing Board of the National Research Council. Papers are refereed by the TRB standing committees identified on page ii of each Record. Reviewers are selected from among committee members and other outside experts. The Transportation Research Board requires a minimum of three reviews; a decision is based on reviewer comments and resultant author revision. For details about the peer review process, see the information on the inside back cover.

THE TRANSPORTATION RESEARCH RECORD PUBLICATION BOARD, comprising a cross section of transportation disciplines and with equal representation from the academic and practitioner communities, assures that the quality of the *Transportation Research Record: Journal of the Transportation Research Board* and the TRB paper peer review process is consistent with that of a peer-reviewed scientific journal. Members from the academic community make decisions on granting tenure; all academic and practitioner members have authored papers published in peer-reviewed journals and all have participated in the TRB peer review process.

TRANSPORTATION RESEARCH RECORD PAPERS ONLINE: The TRR Journal Online website provides electronic access to the full text of papers that have been published in the Transportation Research Record series since 1996. The site is updated as new Record papers become available. To search abstracts and to find subscription and pricing information, go to www.TRB.org/TRROnline.

TRANSPORTATION RESEARCH BOARD PUBLICATIONS may be ordered directly from the TRB Business Office, through the Internet at www.TRB.org, or by annual subscription through organizational or individual affiliation with TRB. Affiliates and library subscribers are eligible for substantial discounts. For further information, contact the Transportation Research Board Business Office, 500 Fifth Street, NW, Washington, DC 20001 (telephone 202-334-3213; fax 202-334-2519; or e-mail TRBsales@nas.edu).

TRANSPORTATION RESEARCH BOARD 2012 EXECUTIVE COMMITTEE*

Chair: Sandra Rosenbloom, Professor of Planning, University of Arizona, Tucson
Vice Chair: Deborah H. Butler, Executive Vice President, Planning, and CIO, Norfolk Southern Corporation, Norfolk, Virginia
Executive Director: Robert E. Skinner, Jr., Transportation Research Board

Victoria A. Arroyo, Executive Director, Georgetown Climate Center, and Visiting Professor, Georgetown University Law Center, Washington, D.C.
J. Barry Barker, Executive Director, Transit Authority of River City, Louisville, Kentucky
William A. V. Clark, Professor of Geography (emeritus) and Professor of Statistics (emeritus), Department of Geography, University of California, Los Angeles
Eugene A. Conti, Jr., Secretary of Transportation, North Carolina Department of Transportation, Raleigh
James M. Crites, Executive Vice President of Operations, Dallas–Fort Worth International Airport, Texas
Paula J. C. Hammond, Secretary, Washington State Department of Transportation, Olympia
Michael W. Hancock, Secretary, Kentucky Transportation Cabinet, Frankfort
Chris T. Hendrickson, Duquesne Light Professor of Engineering, Carnegie Mellon University, Pittsburgh, Pennsylvania
Adib K. Kanafani, Professor of the Graduate School, University of California, Berkeley (Past Chair, 2009)
Gary P. LaGrange, President and CEO, Port of New Orleans, Louisiana
Michael P. Lewis, Director, Rhode Island Department of Transportation, Providence
Susan Martinovich, Director, Nevada Department of Transportation, Carson City
Joan McDonald, Commissioner, New York State Department of Transportation, Albany
Michael R. Morris, Director of Transportation, North Central Texas Council of Governments, Arlington (Past Chair, 2010)
Tracy L. Rosser, Vice President, Regional General Manager, Wal-Mart Stores, Inc., Mandeville, Louisiana
Henry G. (Gerry) Schwartz, Jr., Chairman (retired), Jacobs/Sverdrup Civil, Inc., St. Louis, Missouri
Beverly A. Scott, General Manager and CEO, Metropolitan Atlanta Rapid Transit Authority, Atlanta, Georgia
David Seltzer, Principal, Mercator Advisors LLC, Philadelphia, Pennsylvania
Kumares C. Sinha, Olson Distinguished Professor of Civil Engineering, Purdue University, West Lafayette, Indiana
Thomas K. Sorel, Commissioner, Minnesota Department of Transportation, St. Paul
Daniel Sperling, Professor of Civil Engineering and Environmental Science and Policy; Director, Institute of Transportation Studies; and Acting Director, Energy Efficiency Center, University of California, Davis
Kirk T. Steudle, Director, Michigan Department of Transportation, Lansing
Douglas W. Stotlar, President and Chief Executive Officer, Con-Way, Inc., Ann Arbor, Michigan
C. Michael Walton, Ernest H. Cockrell Centennial Chair in Engineering, University of Texas, Austin (Past Chair, 1991)

Rebecca M. Brewster, President and COO, American Transportation Research Institute, Smyrna, Georgia (ex officio)
Anne S. Ferro, Administrator, Federal Motor Carrier Safety Administration, U.S. Department of Transportation (ex officio)
LeRoy Gishi, Chief, Division of Transportation, Bureau of Indian Affairs, U.S. Department of the Interior, Washington, D.C. (ex officio)
John T. Gray II, Senior Vice President, Policy and Economics, Association of American Railroads, Washington, D.C. (ex officio)
John C. Horsley, Executive Director, American Association of State Highway and Transportation Officials, Washington, D.C. (ex officio)
Michael P. Huerta, Acting Administrator, Federal Aviation Administration, U.S. Department of Transportation (ex officio)
David T. Matsuda, Administrator, Maritime Administration, U.S. Department of Transportation (ex officio)
Michael P. Melaniphy, President and CEO, American Public Transportation Association, Washington, D.C. (ex officio)
Victor M. Mendez, Administrator, Federal Highway Administration, U.S. Department of Transportation (ex officio)
Tara O'Toole, Under Secretary for Science and Technology, U.S. Department of Homeland Security (ex officio)
Robert J. Papp (Adm., U.S. Coast Guard), Commandant, U.S. Coast Guard, U.S. Department of Homeland Security (ex officio)
Cynthia L. Quarterman, Administrator, Pipeline and Hazardous Materials Safety Administration, U.S. Department of Transportation (ex officio)
Peter M. Rogoff, Administrator, Federal Transit Administration, U.S. Department of Transportation (ex officio)
David L. Strickland, Administrator, National Highway Traffic Safety Administration, U.S. Department of Transportation (ex officio)
Joseph C. Szabo, Administrator, Federal Railroad Administration, U.S. Department of Transportation (ex officio)
Polly Trottenberg, Assistant Secretary for Transportation Policy, U.S. Department of Transportation (ex officio)
Robert L. Van Antwerp (Lt. General, U.S. Army), Chief of Engineers and Commanding General, U.S. Army Corps of Engineers, Washington, D.C. (ex officio)
Barry R. Wallerstein, Executive Officer, South Coast Air Quality Management District, Diamond Bar, California (ex officio)
Gregory D. Winfree, Acting Administrator, Research and Innovative Technology Administration, U.S. Department of Transportation (ex officio)

* Membership as of October 2012.

TRB

TRANSPORTATION RESEARCH RECORD

Journal of the Transportation Research Board, No. 2298

Traffic Control Devices, Visibility, and Highway–Rail Grade Crossings 2012

A Peer-Reviewed Publication

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

Washington, D.C.
2012

www.TRB.org

Transportation Research Record 2298
ISSN 0361-1981 (print)
ISSN 2169-4052 (online)
ISBN 978-0-309-22334-8

Subscriber Categories
Operations and traffic management; safety and human factors; rail

Printed in the United States of America

TRANSPORTATION RESEARCH RECORD PUBLICATION BOARD
C. Michael Walton, PhD, PE, Ernest H. Cockrell Centennial Chair in Engineering and Professor of Civil Engineering, University of Texas, Austin (Cochair)
Mary Lynn Tischer, PhD, Director, Office of Transportation Policy Studies, Federal Highway Administration, Washington, D.C. (Cochair)
Daniel Brand, SB, SM, PE, Consultant, Lyme, New Hampshire
Mary R. Brooks, BOT, MBA, PhD, William A. Black Chair of Commerce, Dalhousie University, Halifax, Nova Scotia, Canada
Charles E. Howard, Jr., MCRP, Director, Transportation Planning, Puget Sound Regional Council, Seattle, Washington
Thomas J. Kazmierowski, PEng, Manager, Materials Engineering and Research Office, Ontario Ministry of Transportation, Toronto, Canada
Michael D. Meyer, PhD, PE, Frederick R. Dickerson Professor, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta
Sandra Rosenbloom, MA, PhD, Professor of Planning, University of Arizona, Tucson
Kumares C. Sinha, PhD, PE, Olson Distinguished Professor, School of Civil Engineering, Purdue University, West Lafayette, Indiana
L. David Suits, MSCE, Executive Director, North American Geosynthetics Society, Albany, New York

Peer Review of Transportation Research Record 2298

OPERATIONS AND PRESERVATION GROUP
Paul J. Carlson, Texas A&M Transportation Institute (Chair)

Operations Section
Peter M. Briglia, Jr., Washington State Department of Transportation (Chair)

Traffic Control Devices Committee
David A. Noyce, University of Wisconsin–Madison (Chair), Timothy Jordan Gates, Wayne State University (Secretary), Ernesto Cipriani, Mark Raymond Crabtree, Kimberly A. Eccles, Melisa D. Finley, H. Gene Hawkins, Jr., Ernest David Levi Huckaby, David S. Hurwitz, Kent Kacir, Michael A. Knodler, Jr., Kenneth F. Kobetsky, Scott O. Kuznicki, Mark J. Nahra, Scott E. Nodes, Jon T. Obenberger, Michael P. Reese, Robert K. Seyfried, Michael Sharkady, Jonathan Upchurch

Highway–Rail Grade Crossings Committee
Stephen C. Laffey, Illinois Commerce Commission (Chair), Jason L. Field, Moffatt & Nichol Engineers (Secretary), William L. Barringer, Terry Byrne, Anya A. Carroll, Marco P. da Silva, Timothy J. DePaepe, Diego B. Franca, Deborah M. Freund, Danny A. Gilbert, William P. Grizard, Suzanne M. Horton, Shou-Ren Hu, Aemal J. Khattak, Charles Raymond Lewis II, Richard A. Mather, J. Reilly McCarren, Young-Jun Moon, Aidan E. C. Nelson, Kirsi Pajunen, Joseph G. Palsa, Phillip R. Poichuk, Annie Protopapas, Richard A. Raub, Robert A. Rescot, Ronald E. Ries, Eugene Robert Russell, Sr., Frank F. Saccomanno, John T. Sharkey, Helen Sramek, Paul C. Worley, Guan Xu

SAFETY AND SYSTEMS USERS GROUP
Ronald R. Knipling, safetyforthelonghaul.com (Chair)

Visibility Committee
Carl K. Andersen, Federal Highway Administration (Chair), Jason A. Simmers, Kittelson & Associates, Inc. (Secretary), Vagn Askjaer, Raouf Babari, Andrea Bill, Darrell S. Bowman, Jerremy E. Clark, Ellie L. Francis, Christopher Gaudette, Ronald B. Gibbons, Enrique Gonzalez-Velez, Santosh Kumar Gupta, Neal R. Hawkins, Vichika Iragavarapu, Wade Odell, D. Alfred Owens, Theodore Anton Petritsch, Frank Schieber, Richard E. Stark, Richard Tyrrell, Kenneth D. Uding

Peer review is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 2011.

Transportation Research Board Staff
Richard A. Cunard, Senior Program Officer and Traffic and Operations Engineer
Freda R. Morgan, Senior Program Associate
Mary Kissi, Senior Program Associate

Publications Office
Susan Zinninger, Editor; Janet M. McNaughton, Production Editor; Kristin C. Sawyer, Proofreader; Peter Dull, Manuscript Preparer

Ann E. Petty, Managing Editor; Juanita Green, Production Manager; Phyllis Barber, Publishing Services Manager; Jennifer J. Weeks, Manuscript Preparation Manager

THE NATIONAL ACADEMIES

Advisers to the Nation on Science, Engineering, and Medicine

The **National Academy of Sciences** is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. On the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Ralph J. Cicerone is president of the National Academy of Sciences.

The **National Academy of Engineering** was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Charles M. Vest is president of the National Academy of Engineering.

The **Institute of Medicine** was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, on its own initiative, to identify issues of medical care, research, and education. Dr. Harvey V. Fineberg is president of the Institute of Medicine.

The **National Research Council** was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Ralph J. Cicerone and Dr. Charles M. Vest are chair and vice chair, respectively, of the National Research Council.

The **Transportation Research Board** is one of six major divisions of the National Research Council. The mission of the Transportation Research Board is to provide leadership in transportation innovation and progress through research and information exchange, conducted within a setting that is objective, interdisciplinary, and multimodal. The Board's varied activities annually engage about 7,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation. **www.TRB.org**

www.national-academies.org

TRB SPONSORS*

Transportation Departments of the 50 States and the District of Columbia

Federal Government
U.S. Department of Transportation
- Federal Aviation Administration
- Federal Highway Administration
- Federal Motor Carrier Safety Administration
- Federal Railroad Administration
- Federal Transit Administration
- National Highway Traffic Safety Administration
- Research and Innovative Technology Administration

Bureau of Indian Affairs
Office of Naval Research, U.S. Navy
Science and Technology Directorate, U.S. Department of Homeland Security
U.S. Army Corps of Engineers
U.S. Coast Guard

Nongovernmental Organizations
American Association of State Highway and Transportation Officials
American Public Transportation Association
American Transportation Research Institute
Association of American Railroads
South Coast Air Quality Management District, California

---

*As of October 2012.

TRANSPORTATION RESEARCH RECORD

Journal of the Transportation Research Board, No. 2298

Contents

Foreword vii

Guidelines for Timing Yellow and Red Intervals at Signalized Intersections 1
Hugh McGee, Sr., Kevin Moriarty, and Timothy J. Gates

Comprehensive Evaluation of Driver Behavior to Establish Parameters for Timing of Yellow Change and Red Clearance Intervals 9
Timothy J. Gates, Hugh McGee, Sr., Kevin Moriarty, and Honey-Um Maria

Closed-Course Evaluation of Stop Paddles with Embedded Lights 22
Melisa D. Finley, Eun Sug Park, and William Huff

Detection, Control, and Warning System for Mitigating Dilemma Zone Problem 30
Karl Zimmerman, Devendra Tolani, Roger Xu, Tao Qian, and Peter Huang

Evaluation of Two Treatments for Reducing Crashes Related to Traffic Signal Change Intervals 38
Bhagwant Persaud, Frank Gross, and Raghavan Srinivasan

Evaluation of Comprehension and Legibility of International and Domestic Nonstandard Symbol Signs 46
Bryan J. Katz, Erin E. Dagnall, and Cara C. O'Donnell

Effect of Train Arrival Time on Crash Frequency at Highway–Railroad Grade Crossings: General Classification Regression Model 61
Shou-Ren Hu and Jhy-Pyng Lin

Evaluating Safety at Railway Level Crossings with Microsimulation Modeling 70
Li-Sian Tey, Inhi Kim, and Luis Ferreira

Effects of Environmental Lighting Conditions on Operating Speeds on Urban Arterials 78
Marco Bassani and Guglielmina Mutani

High-Beam Usage on Low-Volume Rural Roads in Texas 88
Vichika Iragavarapu and Kay Fitzpatrick

Development of Visual Model for Exploring Relationship Between Nighttime Driving Behavior and Roadway Visibility Features 96
Ronald B. Gibbons, Christopher J. Edwards, Rajaram Bhagavathula, Paul Carlson, and D. Alfred Owens

Foreword

The 2012 series of the *Transportation Research Record: Journal of the Transportation Research Board* consists of approximately 940 papers selected from more than 4,000 submissions after rigorous peer review. The peer review for each paper published in this volume was coordinated by the committee acknowledged at the end of the text; members of the reviewing committees for the papers in this volume are listed on page ii.

Additional information about the *Transportation Research Record: Journal of the Transportation Research Board* series and the peer review process appears on the inside back cover. TRB appreciates the interest shown by authors in offering their papers, and the Board looks forward to future submissions.

Note: Many of the photographs, figures, and tables in this volume have been converted from color to grayscale for printing. The electronic files of the papers, posted on the web at **www.TRB.org/TRROnline**, retain the color versions of photographs, figures, and tables as originally submitted for publication.

Measurement Conversion Factors

To convert from the unit in the first column to the unit in the second column, multiply by the factor in the third column.

Customary Unit	*SI Unit*	*Factor*
Length		
inches	millimeters	25.4
inches	centimeters	2.54
feet	meters	0.305
yards	meters	0.914
miles	kilometers	1.61
Area		
square inches	square millimeters	645.1
square feet	square meters	0.093
square yards	square meters	0.836
acres	hectares	0.405
square miles	square kilometers	2.59
Volume		
gallons	liters	3.785
cubic feet	cubic meters	0.028
cubic yards	cubic meters	0.765
Mass		
ounces	grams	28.35
pounds	kilograms	0.454
short tons	megagrams	0.907
Illumination		
footcandles	lux	10.76
footlamberts	candelas per square meter	3.426
Force and Pressure or Stress		
poundforce	newtons	4.45
poundforce per square inch	kilopascals	6.89
Temperature		

To convert Fahrenheit temperature (°F) to Celsius temperature (°C), use the following formula:
°C = (°F − 32)/1.8

SI Unit	*Customary Unit*	*Factor*
Length		
millimeters	inches	0.039
centimeters	inches	0.394
meters	feet	3.281
meters	yards	1.094
kilometers	miles	0.621
Area		
square millimeters	square inches	0.00155
square meters	square feet	10.764
square meters	square yards	1.196
hectares	acres	2.471
square kilometers	square miles	0.386
Volume		
liters	gallons	0.264
cubic meters	cubic feet	35.314
cubic meters	cubic yards	1.308
Mass		
grams	ounces	0.035
kilograms	pounds	2.205
megagrams	short tons	1.102
Illumination		
lux	footcandles	0.093
candelas per square meter	footlamberts	0.292
Force and Pressure or Stress		
newtons	poundforce	0.225
kilopascals	poundforce per square inch	0.145
Temperature		

To convert Celsius temperature (°C) to Fahrenheit temperature (°F), use the following formula:
°F = (°C × 1.8) + 32

Abbreviations Used Without Definitions

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACRP	Airport Cooperative Research Program
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials (known by abbreviation only)
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISO	International Organization for Standardization
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SHRP	Strategic Highway Research Program
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board

Guidelines for Timing Yellow and Red Intervals at Signalized Intersections

Hugh McGee, Sr., Kevin Moriarty, and Timothy J. Gates

The lack of a national standard, recommended practice, or set of guidelines for determining the duration of the yellow change and red clearance intervals for the timing of traffic signals has left each transportation agency to determine its own practices. The objective of NCHRP Project 03-95 was to develop a comprehensive and uniform set of recommended guidelines for determining safe and operationally efficient yellow and red intervals at signalized intersections. To accomplish this objective, the study established a state of knowledge and a state of practice through a review of existing guidelines and various literature sources and a survey of transportation agency practitioners. The research also conducted a comprehensive national investigation of driver behavioral characteristics through an extensive field data collection and analysis effort. These efforts confirmed the accepted values for perception–reaction time (1.0 s) and deceleration rate (10 ft/s^2) and established 85th percentile speed estimations for through and left-turning vehicles. Justification was also provided for accounting for start-up delay. From the findings, a succinct recommended guideline for the timing of yellow change and red clearance intervals was formulated on the basis of a kinematic equation and its associated variable values. The recommended guideline encourages a uniform practical application that provides a framework that can be easily adopted into transportation agency practice.

The function of a traffic signal is to alternate the right-of-way among various traffic movements. The signal must accomplish this process in a safe manner while minimizing intersection user delay and maximizing intersection capacity. Vital to accomplishing these goals is the yellow change interval. As stated in the *Manual on Uniform Traffic Control Devices* (MUTCD), the yellow signal indication "warns traffic of an impending change in the right-of-way assignment" (*1*). Drivers approaching the intersection during the yellow interval must decide to stop or continue through the intersection.

In many jurisdictions, the yellow change interval is followed by a red clearance interval (also referred to as an "all-red interval"). During the red interval, the red signal indication is displayed to all potentially conflicting traffic movements. Its purpose is "to provide additional time before conflicting traffic movements, including pedestrians, are released" (*1*).

H. McGee, Sr., and K. Moriarty, Vanasse Hangen Brustlin, Inc., 8300 Boone Boulevard, Suite 700, Vienna, VA 20165. T. J. Gates, Department of Civil and Environmental Engineering, Wayne State University, 5050 Anthony Wayne Drive, Detroit, MI 48202. Corresponding author: K. Moriarty, kmoriarty@vhb.com.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 1–8.
DOI: 10.3141/2298-01

The selection of an appropriate yellow interval length and the decision whether to employ a red clearance interval are important to both safety and capacity. However, despite the importance of these intervals, there is no single national standard, recommended practice, or set of guidelines for calculating their duration. This gap leaves transportation agencies responsible for determining and defending their own practices.

In recognition of this and given that the key parameters involved in calculating yellow and red intervals—specifically, driver reaction times and vehicle deceleration rates—were determined more than 25 years ago, NCHRP Project 03-95 (Guidelines for Timing Yellow and Red Intervals at Signalized Intersections) was conducted. This research was based on the premise that these parameters should be updated and that other important factors in calculating interval duration should be considered, including speeds, grades, vehicle types and mix, road surface conditions, geometry, coordinated systems and isolated signals, signal timing parameters, driver age, and turning movements.

OBJECTIVE

This paper summarizes the key findings of NCHRP Project 03-95 and presents the recommended guidelines formulated from this effort. The objective of this research was to develop a comprehensive and uniform set of recommended guidelines for determining safe and operationally efficient yellow and red intervals at signalized intersections. This objective was accomplished by

- Developing a comprehensive inventory of factors to be considered in policies and procedures for calculating and implementing yellow and red intervals;
- Critically reviewing and summarizing literature relevant to factors potentially affecting change intervals and examining rational approaches to calculating yellow and red intervals;
- Surveying the yellow and red timing practices of public agencies;
- Reviewing past studies and agency operational experiences relating change interval timing to crashes caused by the running of red lights;
- Conducting field studies at a diverse sample of intersections to obtain data on critical parameters, including reaction time, deceleration rate, and approach speed; and
- Synthesizing the field study results along with other information to formulate a recommended guideline for determining yellow and red intervals.

This project was conducted in parallel with a similar effort by ITE in its development of a recommended practice.

EXISTING RESOURCES

Several national publications have served as references for yellow and red interval timing:

- FHWA *Manual on Uniform Traffic Control Devices* (*1*),
- ITE *Traffic Engineering Handbook* (*2*),
- ITE *Manual of Traffic Signal Design* (*3*),
- ITE *Traffic Control Devices Handbook 2001* (*4*),
- FHWA *Signalized Intersections: Informational Guide* (*5*),
- FHWA *Traffic Signal Timing Manual* (*6*), and
- FHWA *Yellow Change Intervals* memorandum (*7*).

The treatment of yellow and red intervals in each of these publications is discussed in the following sections.

Manual on Uniform Traffic Control Devices

The MUTCD is the primary document for traffic control device standards, including traffic signalization (*1*). The intent of the MUTCD is to establish uniformity for all traffic control devices so that users encounter consistent applications across the nation. Section 4D.26 provides limited standards and guidance regarding yellow and red timing. The MUTCD prescribes that practitioners use the ITE *Manual of Traffic Signal Design* or the ITE *Traffic Control Devices Handbook* to determine the duration of yellow and red intervals. Additionally, the guidance states that the duration of the yellow change interval should be a minimum of 3 s and a maximum of 6 s and that the duration of the red clearance interval should be a maximum of 6 s.

ITE and FHWA Publications

Beyond the MUTCD, practitioners often look to ITE for guidance on traffic engineering issues. In 1985, ITE developed a proposed recommended practice that was based on the following kinematic equation for calculating the vehicle clearance interval, or change period (*8*):

$$\mathrm{CP} = t + \frac{V}{2a + 64.4g} + \frac{W + L}{V} \tag{1}$$

where

CP = change period (s),
t = perception–reaction time (s),
a = deceleration rate (ft/s^2),
V = approach speed (ft/s),
g = percentage of grade divided by 100 (positive for upgrade, negative for downgrade),
W = width of intersection (ft), and
L = length of vehicle (ft).

Although ITE never formally adopted this proposed recommended practice, many agencies use it today. The equation provides time for a driver to perceive and react to the yellow indication and either decelerate comfortably to a stop or continue through the intersection before the change of right-of-way. Generally, the first two terms of Equation 1 are used to calculate the yellow change interval; the third term is used to calculate the red clearance interval.

Here are a few noteworthy points about the equations and information provided in the reviewed existing resources:

- The *Traffic Engineering Handbook* does not specify the approach speed measure to be used (e.g., posted speed limit, 85th percentile speed) (*2*). Instead, design speed is referenced without further explanation. The handbook does state that consideration should be given to the 15th percentile speed, particularly at wider intersections. The other national resource publications reviewed for this research recommend using the 85th percentile approach speed. If this is unknown, then the approach speed limit should be substituted for the approach speed. However, the *Yellow Change Intervals* memorandum states that, in the absence of the 85th percentile approach speed, the approach speed limit plus 10 miles per hour (mph) should be used (*7*).
- The *Traffic Control Devices Handbook 2001* (*4*)
  - Defines intersection width as the full intersection width plus the crosswalk widths;
  - Assumes vehicle length to be 15 ft rather than 20 ft; and
  - Provides a modified red clearance interval equation, subtracting 1.0 s to account for the delay in reaction time experienced by drivers in conflicting traffic and their position relative to a conflict point.
- The *Manual of Traffic Signal Design* (*3*) and the *Signalized Intersections: Informational Guide* (*5*) present only the single standard equation approach rather than separate equations for yellow and red intervals. However, the *Manual of Traffic Signal Design* does acknowledge that many agencies set the yellow interval equal to the first two terms of the equation. Likewise, the *Informational Guide* notes that some agencies use the third term of the equation to calculate the duration of the red interval.
- The *Traffic Signal Timing Manual* presents only the single standard equation approach rather than separate equations for yellow and red intervals (*6*). The discussion reiterates that the use of a red interval is optional, and that there is no consensus regarding its application or duration. The manual notes that the third term is to be used to determine the duration of the red interval; however, it is not prescriptive with regard to values for intersection width, vehicle length, or speed, citing "local policy" or "engineering judgment."

STATE VEHICLE CODE AND TIMING PRACTICES OF YELLOW CHANGE AND RED CLEARANCE INTERVALS

Each state determines the law regarding the entry of vehicles into an intersection on the yellow interval. The laws pertaining to yellow intervals may be categorized as either permissive or restrictive. Under a permissive yellow law, drivers may enter the intersection during the entire duration of the yellow interval and legally be in the intersection while the red signal indication is displayed, as long as entrance occurred before or during the yellow signal indication. In permissive applications, the red interval provides additional time as an added safety factor before a change in right-of-way. Under a restrictive yellow law, (*a*) drivers may not enter the intersection during the yellow signal indication unless it can be cleared before a red signal indication, or (*b*) drivers may not enter the intersection unless it is impossible or unsafe to stop. States following the second condition of the restrictive yellow law are generally not in conflict with the permissive yellow law. A review of state vehicle codes revealed

that a true restrictive yellow law (i.e., the first restrictive condition) is followed by four states: Louisiana, Rhode Island, Tennessee, and West Virginia. Of the remaining states, 37 follow a permissive yellow law and nine follow the second condition of the restrictive yellow law.

The interval timing should concur, theoretically, with the law. This becomes important when the ITE equation is used to calculate yellow and red signal durations, particularly with respect to allocation of the change and clearance times. In permissive applications, the first two terms of the ITE equation are typically used to calculate the yellow interval; they are applied to provide enough time for vehicles to enter the intersection but not to clear the far side of the intersection. The necessary clearance time is the red interval, which is calculated with the third equation term. In restrictive applications, all three terms of the ITE equation should, in principle, be used to calculate the yellow interval. A restrictive yellow law does not necessitate the use of a separate red interval, but it does rely on the use of good engineering judgment (*6*).

FIELD STUDY DATA COLLECTION AND ANALYSIS

Developing practical timing guidelines requires an understanding of the behavioral characteristics of drivers approaching an intersection during yellow and red intervals. This project's field investigation focused on three issues surrounding the use of the ITE kinematic equation for interval timing:

- Perception–reaction time,
- Deceleration rate, and
- Approach speeds (through and left-turning vehicles).

To provide a comprehensive set of driver behavioral characteristics, the field investigation captured data from more than 80 intersection approaches in five different states (California, Florida, Maryland, Michigan, and Virginia). The states were selected in different regions of the country to provide the regional diversity necessary to account for variations in timing practices as well as location-specific factors. More than 320 h of video were obtained, from which approximately 7,500 vehicles were extracted as data points for the evaluation portion of this study.

A multifactor analysis of variance performed with SPSS software was used to analyze the data. The independent variables included site-related characteristics and the site-sampling variables. The dependent variables included perception–reaction time for first-to-stop vehicles, deceleration rate for first-to-stop vehicles, and approach speeds for both through vehicles and left-turning vehicles. The field study methodology and findings are thoroughly discussed in NCHRP Report 731, *Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections* (*9*), and are the subject of Gates et al. (*10*).

DISCUSSION OF STUDY RESULTS

What Model Should Be Followed?

Over time, several procedures or models have been proposed and adopted to determine yellow and red interval durations. The MUTCD requires that the duration of yellow change and red clearance intervals "shall be determined using engineering practices" (*1*). The ITE kinematic equation is based on principles of physics and intersection characteristics; therefore, it is the most defensible and adaptable method used in practice. Furthermore, a nationwide state-of-the-practice survey found that this kinematic equation is the most widely used procedure for calculating yellow and red intervals by transportation agencies in the United States. It is for these reasons that the authors recommend using this equation to calculate yellow and red interval durations. A yellow change interval computed with the kinematic equation will provide enough time for a vehicle traveling at or below the assumed prevailing speed to comfortably decelerate to a stop or enter the intersection before the red signal indication is displayed. Similarly, a red clearance interval timed on the basis of this formula will provide additional time for a vehicle that has entered the intersection at the last moment at which the signal is yellow to avoid conflict with vehicles and pedestrians being released from an adjacent opposing intersection approach.

The standard ITE kinematic model has had few changes since its adoption in the *Traffic Engineering Handbook* in 1965 (*11*). A modification factor to accommodate approach grade was incorporated in the 1982 *Manual of Traffic Signal Design* (*3*) and has since been the proposed method in subsequent editions of the *Traffic Engineering Handbook*. The application of the kinematic equation, particularly the allocation of the clearance interval time to either the yellow or the red, is dependent on the state vehicle code.

Should Timing Practices Vary on the Basis of State Vehicle Code?

As stated earlier, there are two basic laws that apply to the meaning of the yellow signal indication and a driver's legal behavior in response to the signal: permissive and restrictive. It is unclear whether drivers are aware of their state vehicle code with respect to the yellow law. Furthermore, it is doubtful that drivers are aware of differing laws or alter their behavior when they travel between jurisdictions.

Restrictive states should provide the entire change and clearance duration to the yellow interval for the signal timing to be in harmony with the vehicle code. From a human factors perspective, providing a longer yellow interval for a prevailing approach speed generally encourages drivers to enter later during the yellow signal (*12*). Thus, providing both the change and clearance durations within the yellow interval may be misleading to drivers who are unfamiliar with the practices of the particular jurisdiction in which they are located. Although a permissive yellow law provides a red interval to allow additional time as a safety factor for vehicles legally within the intersection before the release of conflicting traffic, a restrictive yellow law does not necessarily provide this added factor of safety. Furthermore, many restrictive jurisdictions allocate interval durations on the basis of typical permissive timings (i.e., yellow as the change interval and red as the clearance interval), thereby creating inconsistencies within the law and contributing to violations. In light of these issues, the authors recommend that yellow and red intervals be calculated and implemented in accordance with the permissive yellow law, following the language of the national *Uniform Vehicle Code* (*13*). The authors also recommend that states that currently follow the restrictive yellow law consider changing their vehicle code to follow the permissive yellow law to provide nationwide uniformity and better alignment with driver expectations.

What Are Recommended Equation and Associated Values?

The findings of this study recommend that the equations shown here as Equations 2 and 3 be used to calculate the yellow change and red clearance intervals, respectively:

$$Y = t + \frac{1.47V}{2a + 64.4g} \tag{2}$$

$$R = \frac{W + L}{1.47V} - 1 \tag{3}$$

where V is the 85th percentile approach speed (mph) and the other variables are as defined for Equation 1. Note that W is measured from the back edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane. The recommended input values for Equations 2 and 3 are provided in the sections that follow.

Yellow Change Interval (Equation 2)

Perception–Reaction Time (t) One second has been the suggested reaction time in the ITE kinematic equation since the 1965 *Traffic Engineering Handbook* (*9*). This constant recognizes that there is a period of time from the moment a driver sees the signal change from green to yellow to when the driver reacts. The reaction and associated time depends on the driver's position relative to the intersection. The driver's reaction can be to (*a*) stay at current speed, continuing into the intersection; (*b*) accelerate, perceiving the necessity to increase speed to legally enter the intersection; or (*c*) decelerate, intending to stop.

Reaction time is a constant in the ITE equation, having a direct one-to-one effect on the calculated interval. This factor was examined for drivers who came to a stop in response to the onset of a yellow signal display. The mean value was found to be exactly 1.0 s with an 85th percentile value of 1.33 s. It was determined that a driver's selection of reaction time and deceleration rate are dependent on one another. Slow-reacting drivers tend to compensate with greater deceleration rates, and quick-reacting drivers tend to decelerate more comfortably. This point is thoroughly addressed by Parsonson (*14*) and was confirmed in the field study performed as part of this research. This research also indicated that most site-related factors had little to no impact on reaction time or deceleration rate. Thus, the authors recommend considering the mean or median value when selecting a reaction time and, as will be discussed, deceleration rate for timing the yellow interval. The authors recommend that 1.0 s be used for the value of reaction time in the kinematic equation.

Deceleration Rate (a) Deceleration rate has the largest effect on the variance of the calculated change interval when the ITE kinematic equation is applied. Therefore, it is important to determine the most appropriate value of this rate when the yellow interval is calculated. The length of the yellow interval is inversely related to the deceleration rate (i.e., as the deceleration rate decreases, the calculated yellow duration increases).

The value of 15 ft/s^2 was assumed in ITE manuals until 1982, when, in the *Manual of Traffic Signal Design* (*3*) and the *Traffic Engineering Handbook* (*15*), it was reduced to the currently assumed value of 10 ft/s^2. A few years before this alteration, Parsonson and Santiago had asserted that 15 ft/s^2 was for emergency stopping distance and should not apply to change interval timing (*16*).

This research found the mean deceleration rate to be 10.08 ft/s^2 with an 85th percentile value of 12.89 ft/s^2. The deceleration rates observed in this study typically varied in the following manner:

- They increased as approach speeds increased (i.e., faster drivers used greater deceleration),
- They decreased as travel times to the intersection at the start of a yellow signal increased (i.e., drivers used lower deceleration when they were farther from the intersection), and
- They increased as the brake response times increased (i.e., drivers who reacted more slowly used greater deceleration rates).

As stated previously, reaction time and deceleration rate were found to be directly correlated with each other with minimal impact from temporally related site factors. Thus, the authors recommend considering the mean or median value when selecting a reaction time and deceleration rate for timing the yellow interval. The authors recommend that a deceleration rate of 10 ft/s^2 be used in the kinematic equation.

Approach Speed (V) The speed variable appears in the equations for both yellow and red intervals. The *Traffic Engineering Handbook* uses the term "design speed" but does not elaborate on the meaning when the term is applied to the equation (*2*). Unless the signal timing is being prepared for a traffic signal on a new road where the design speed is known, this speed metric is typically unavailable. Also, the actual operating speed, specifically, the 85th percentile speed, is typically different from the design speed.

Previous editions of the *Traffic Engineering Handbook* and other national resource publications recommend the 85th percentile speed, although some references recommend using the speed limit. The selection of the 85th percentile as the design specification has ample precedent in traffic engineering. The 85th percentile speed is based on the premise that the vast majority of drivers will select a speed that is reasonable, safe, and prudent for a given road. The 85th percentile approach speed is the preferred variable measurement to use in the ITE equation, as it captures 85% of the drivers traveling at or below the value. However, its use typically requires the performance of a speed study to establish prevailing conditions, something that is not always practical.

This research found that, in practice, speed limit is more often used as the design specification because it is more easily identified than the 85th percentile speed. Although the latter typically provides the basis for establishing speed limits, it is commonly acknowledged that the actual 85th percentile speed of a roadway typically exceeds the speed limit. The findings of the field study showed that the overall 85th percentile speed was, on average, 7.45 mph greater than the posted speed limit. On a site-by-site basis, the 85th percentile speeds were, on average, 6.49 mph greater than the posted speed limit. Thus, it was concluded that the speed limit typically does not provide an accurate estimate of the 85th percentile speed.

With respect to timing practice, it is desirable to keep operating speed recommendations as simple as possible—preferably based on the posted speed limit plus a constant value. Thus, in lieu of field-measured speed data, the authors recommend using the approach speed limit plus 7 mph as a rule-of-thumb estimate for the 85th percentile through-movement approach speed.

The field study also investigated approach speeds for left-turning vehicles. Left-turning drivers must decelerate to navigate the turn safely; the turn itself is not typically a direct response to the yellow signal indication. Approaches with speed limits of 40 to 50 mph showed the overall mean left-turn approach speed to be 10.59 mph less than the posted speed limit. The overall 85th percentile left-turn approach speed was 4.94 mph less than the posted speed limit. Locations with posted speed limits of 35 mph or less were not investigated, as the operating speed for left-turning vehicles on low-speed approaches was assumed to be the same (or nearly so) value as that for through vehicles. For all approaches, the authors recommend using the approach speed limit minus 5 mph to calculate the left-turn yellow interval duration.

Grade (g) Grade was introduced to the kinematic equation in 1982 and has been used ever since. The literature indicates that, for every 1% upgrade, the calculated duration of the yellow interval is decreased by 0.1 s. Conversely, for every 1% downgrade, the calculated duration of the yellow interval is increased by 0.1 s.

Steep downgrades had an impact on reaction time as drivers began braking before the onset of the yellow interval, helping to keep deceleration rates at comfortable levels. Sites with upgrades greater than 3% exhibited deceleration rates that were much higher than those on level grade, whereas sites with downgrades greater than 3% exhibited deceleration rates that were much lower than those on level grade.

The authors do not recommend that grade-related adjustments be made to reaction time and deceleration rate for use in calculation of the yellow change interval. However, approach grade should continue to be accounted for explicitly in the kinematic equation. As a general rule, the grade measurement should be taken at the distance corresponding to the upper boundary of the dilemma zone (i.e., approximately 5.0 s upstream of the stop line), as based on the approach speed limit plus 7 mph.

Red Clearance Interval (Equation 3)

Approach Speed (V) Some timing procedures or methods assume a different approach speed when determining the red interval. Instead of the same 85th percentile approach speed as for the yellow interval, use of the 15th percentile or even 10th percentile approach speed is suggested (*17*). Slower-traveling vehicles will be exposed to or create potential conflict longer than faster-traveling vehicles, and therefore they would require more red clearance time. However, applying the lower-speed threshold would be overly conservative and inappropriate, as slower vehicles will generally be provided with enough yellow time to stop comfortably. Therefore, the authors recommend using the same speed value (i.e., 85th percentile) to calculate the red clearance interval for through vehicles as was used to calculate the yellow change interval.

Turning-vehicle speeds within the intersection were not collected, and therefore data cannot be provided. However, with the use of the AASHTO equation for calculating the design speed of horizontal curves (*18*), the average left-turn design speed for the study sites was calculated as 16.3 mph with an 85th percentile value of 18.5 mph. The literature suggests that left-turn maneuvers are typically performed at speeds between 15 and 25 mph, depending on the turning radius. Therefore, the authors recommend using the value of 20 mph to calculate the duration of the left-turn red interval, regardless of the approach speed limit.

Length of Vehicle (L) The length of the vehicle used in the calculation of the red interval typically ranges from 15 to 20 ft. AASHTO specifies a design-vehicle passenger car as having a length of 19 ft (*18*). Arguments have been made to consider larger vehicles, such as single-unit trucks (30 ft) or intermediate semitrailers (55 ft), in the timing calculations. A consideration of larger vehicles would increase the duration of the red interval to accommodate the additional vehicle length before conflicting traffic was released. However, conflicting vehicular traffic is obligated to yield the right-of-way to other vehicles legally in the intersection (*1*), which would include truck trailers. The length of the vehicle is irrelevant to this requirement. Therefore, the authors recommend using a vehicle length of 20 ft to calculate the red clearance interval.

Intersection Width (W) The *Traffic Engineering Handbook* defines intersection width as the distance from the stop line to the no-conflict point on the far side of the intersection (*2*). The other national resource publications reviewed for this research suggest the following options, listed in order of shortest to longest distance:

1. Curb to curb (or the extension of the travel edges of the conflicting roads),
2. Nearside stop line to the middle of the first conflicting traffic lane,
3. Nearside stop line to the far edge of the last conflicting traffic lane,
4. Nearside stop line to farside curb, and
5. Nearside stop line to far side of the crosswalk, if one exists.

Conservatively, Option 5 suggests that the intersection width used to calculate the red clearance interval should extend to the far side of the crosswalk, ensuring that pedestrians are not released until conflicting vehicles clear the crosswalk. A liberal position would argue that a pedestrian would be cognizant of a vehicle approaching the crosswalk area and would not enter into the path of the oncoming vehicle. In situations involving blind or visually impaired pedestrians, auditory cues are used to establish vehicle presence, direction of travel, and acceleration rate or speed (*19*). With these cues, blind or visually impaired pedestrians would likely be aware of an oncoming vehicle and would not enter the crosswalk. However, safely accommodating this user group is often a challenge, as intersection geometries, signal equipment, vehicle characteristics, and driver behavior vary greatly.

If Options 1 through 4 were considered, the conservative approach would be to choose Option 4 to ensure that potential conflicting vehicles make it through the intersection. The more liberal approach would be Option 2, to accommodate conflicting vehicles from the stop line to the middle of the first conflict lane (likely the left-turn lane). The argument for this option is that drivers on the conflicting approach would see a vehicle in the intersection and would be obligated by law to wait to enter until the vehicle had passed.

Clearing width with respect to the presence of crossings equipped with pedestrian signals should be considered on receiving lanes. Although a pedestrian is legally obligated to yield the right-of-way to a vehicle lawfully in the intersection, as detailed in Section 4E.02 Standard A of the MUTCD (*1*) and in accordance with the *Uniform Vehicle Code* (*13*), engineers maintain that a factor of safety should be considered to eliminate the possibility of pedestrian–vehicle conflict. As already suggested, a conservative approach would be to include the entire width of the crossing.

Pedestrians, like drivers, exhibit a start-up delay when they react to the change of a pedestrian signal indication from "Don't Walk"

to "Walk." This delay is defined as the time between the instant at which the "Walk" signal becomes illuminated and the instant at which the pedestrian first steps off the curb. Knoblauch et al. indicate mean start-up times of 1.93 s and 2.48 s for younger and older pedestrians, respectively (*20*). The 85th percentile times are reported as 3.06 s and 3.76 s for younger and older pedestrians, respectively. The *Highway Capacity Manual* recommends a design value of 3 s (*21*).

If the smallest value of 1.93 s for a younger pedestrian is assumed, then a vehicle approaching at a 25 mph posted speed limit plus 7 mph (total speed of 47 ft/s) will traverse approximately 91 ft during this time. If a 1-s reduction of the red interval to account for vehicular start-up delay is considered (as discussed next), then the effective pedestrian start-up delay becomes 0.93 s. For the same approach speed conditions, a vehicle will traverse approximately 43 ft during this time. These conditions account for the worst-case scenario (i.e., the quickest pedestrian reaction time and slowest vehicular approach speed). Faster speeds will traverse longer distances in the same period, thus clearing vehicle–pedestrian conflicts. The presence of a pedestrian crossing should only be a concern if pedestrian signals are present, as the pedestrian signal indicates when it is permissible to leave the curb.

The authors recommend the intersection width be measured from the back edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane. For left-turn movements, the measurement would be taken along the turning path of the approaching vehicle. A pedestrian crossing equipped with pedestrian signals on a receiving lane should not be considered unless the nearest crossing line is 40 ft or more from the extension of the farthest edge of the farthest conflicting traffic lane. If this condition exists, then the intersection width should be measured to the nearest pedestrian crossing line.

Reduction (−1 s) Drivers in lead vehicles at the stop line typically exhibit delay when reacting to the display of a green signal. The field study evaluated start-up delay and determined an average value of 1.1 s. Similarly, total intersection entry delay to the nearest conflicting lane was evaluated and determined to be, on average, 4.1 s. On the basis of these findings, it was concluded that a 1-s reduction should be included in calculating the red clearance interval. This value is conservative compared with a 4-s reduction when the additional factors that delay intersection entry are considered.

Reducing the calculated red clearance interval by 1 s has the potential to create situations in which a driver may not be entirely clear of the intersection before the beginning of the next conflicting phase. This may be a concern for corridors with a progressive signal system, as drivers become familiar with signal coordination and enter an intersection at speed as the signal display turns to green (also referred to as "gaming"). However, drivers entering an intersection from an opposing approach under the green signal indication are obligated to yield the right-of-way to other traffic legally within the intersection (*1*). Essentially, the 1-s reduction redefines intersection clearance, recognizing that an intersection need be only partially cleared and that the start-up delay and spatial buffers provide for further intersection clearance before potential conflict.

Should There Be Minimum and Maximum Interval Values?

As previously stated, the MUTCD provides guidance with regard to the minimum and maximum durations of the yellow and red clearance intervals. This study did not reveal any support for these suggested values. Presumably, the rationale is to prevent extremely short or excessively long durations, which may not provide an adequate level of safety or may be counterproductive to efficient intersection operations. When yellow and red intervals are implemented, there is a trade-off between intersection safety and intersection operations.

The authors find no reason to suggest minimum or maximum values for the yellow interval. For the red interval, the authors suggest using a minimum of 1.0 s to provide additional time to vehicles legally within the intersection as a factor of safety before the release of conflicting traffic from an adjacent opposing intersection approach. Anything less than 1 s would seem unreasonable; intuitively, a fraction of a second is not an amount of time that provides the benefit of any added safety factor. A maximum red clearance interval is not suggested, however. Driver behavior and intersection conditions that result in long red intervals are atypical; however, if the calculated red interval is excessively long, then engineering judgment should be used to determine the appropriate application.

Is Red Clearance Interval Necessary?

According to the MUTCD, the use of a red clearance interval is optional (*1*). Vehicular traffic presented with a green signal display must yield the right-of-way to other vehicular traffic and pedestrians lawfully within the intersection or crosswalk. Because the provision of a red interval is considered a safety factor, one would assume a reduction in the occurrence of angle crashes. However, this research effort indicated that previously published empirical studies have not consistently or definitively demonstrated long-term crash reductions. Therefore, not providing red intervals should not necessarily be interpreted as being detrimental to safety. The authors recommend, however, that the yellow change and red clearance intervals be calculated in accordance with the permissive law; therefore, a red clearance interval is appropriate.

Left-Turn Change Intervals

When yellow change and red clearance intervals are calculated for left-turning vehicles, signal phasing should be considered as follows:

- For protected left-turn movements, the yellow and red intervals should be calculated for each approach and implemented as calculated. The intervals do not have to be the same duration for opposing approaches.
- For permissive left-turn movements, the yellow and red intervals should be calculated for opposing approaches, including the through movements. The implemented intervals should be the longest of the calculated values (left, through, or combination). The intervals should be the same duration for the left-turn and through movements on opposing approaches to ensure that termination is concurrent.
- For protected–permissive left-turn movements, the yellow and red intervals should be calculated and implemented as already described for the respective protected and permissive portions of the phase.

RECOMMENDED GUIDELINE

Fundamentally, all traffic control devices should be applied in a uniform manner so that drivers experience the same device and its operation regardless of location. This would pertain to timing yellow

change and red clearance intervals. To maintain this fundamental principle, the recommended guideline strives to achieve national acceptance for a uniform practical application. It provides a solid framework, based on research and accepted practice, that is succinct in scope, requires little user interpretation, and can be easily adopted into transportation agency practice.

Although the guidelines are based on typical roadway and driver conditions, there may be instances when exceptions are necessary to accommodate unusual conditions. Under these circumstances, engineering judgment may be exercised to use other implementation practices. However, under typical roadway and driver conditions, drivers should expect the duration of yellow and red intervals to be calculated and implemented according to the recommended kinematic equation and its associated recommended values.

The yellow change and red clearance intervals should be calculated with the equations and associated parameters presented in the following sections.

Yellow Change Interval

Equation 4 should be used to calculate the yellow change interval (Y):

$$Y = t + \frac{1.47V}{2a + 64.4g} \tag{4}$$

where V is the 85th percentile approach speed (mph) determined under free-flow conditions and the other variables are as defined above. The value recommended for t is 1.0 s and the value recommended for a is 10 ft/s^2. If the 85th percentile approach speed for through movements is available, then the yellow interval is calculated directly from Equation 4. Because the 85th percentile speed is typically unavailable, it can be assumed as the approach speed limit plus 7 mph, except for left-turn movements (as explained). Table 1 provides yellow intervals based on typical roadway and driver conditions; it assumes the posted speed limit plus 7 mph for grades in the range of ±4%.

Red Clearance Interval

Equation 5 should be used to calculate the red clearance interval (R):

$$R = \frac{W + L}{1.47V} - 1 \tag{5}$$

TABLE 1 Yellow Change Interval(s) for Through Movements by Approach Speed Limit and Grade

Posted Speed Limit (mph)[a]	Grade (%)				
	–4	–2	0	2	4
25	3.7	3.5	3.4	3.2	3.1
30	4.1	3.9	3.7	3.6	3.4
35	4.5	4.3	4.1	3.9	3.7
40	5.0	4.7	4.5	4.2	4.1
45	5.4	5.1	4.8	4.6	4.4
50	5.8	5.5	5.2	4.9	4.7
55	6.2	5.9	5.6	5.3	5.0

[a]All posted speed limits increased by 7 mph to estimate 85th percentile speed for calculating yellow change interval.

where V is the 85th percentile approach speed (mph) and L is set at 20 ft. As in Equation 3, W should be measured from the back edge of the approaching movement stop line to the far side of the intersection limit as determined by the extension of the curb line or outside edge of the farthest travel lane. A pedestrian crossing equipped with pedestrian signals on a receiving lane should not be considered unless the nearest crossing line is 40 ft or more from the extension of the farthest edge of the farthest conflicting traffic lane. If this condition exists, then the intersection width should be measured to the nearest pedestrian crossing line. The same approach speed value used to calculate the yellow interval for through movements should be used to calculate the red interval, except for left-turn movements (as explained). The reduction of 1 s is to account for the start-up delay typically incurred by a driver stopped on a conflicting approach to react to a green signal indication.

The following provisions apply for the duration of a calculated red interval:

- If the calculated red interval is less than or equal to 1.0 s, then the minimum design should be 1.0 s.
- If the calculated red interval is greater than 1.0 s, then the duration should be as calculated.

Left-Turn Movements

Equations 4 and 5 should be used to calculate yellow change and red clearance intervals for left-turn movements, with the use of the following modified parameters:

- For the yellow change interval, V is still the approach speed (mph), but it should be set at the approach speed limit minus 5 mph.
- For the red clearance interval, W is the length (ft) of the approaching vehicle turning path (see previous discussion); V is still the approach speed (mph), but it should be set at 20 mph regardless of the approach speed limit.

When yellow change and red clearance intervals are calculated for left-turning vehicles, signal phasing should be considered as follows:

- For protected left-turn movements, the yellow and red intervals should be calculated for each approach and implemented as calculated. The intervals do not have to be the same duration for opposing approaches.
- For permissive left-turn movements, the yellow and red intervals should be calculated for opposing approaches, including the through movements. The implemented intervals should be the longest of the calculated values (left, through, or combination). The intervals should be the same duration for the left-turn and through movements on opposing approaches to ensure that termination is concurrent.
- For protected–permissive left-turn movements, the yellow and red intervals should be calculated and implemented as already described for the respective protected and permissive portions of the phase.

Other Considerations: Grade Measurement

If approach grade is applicable, as a general rule, measurements should be taken at the distance corresponding to the upper boundary of the dilemma zone (i.e., approximately 5.0 s upstream of the stop line) based on the approach speed limit plus 7 mph.

CONCLUSION

This research confirmed that agencies responsible for the timing of change intervals take a widely varied approach in their practices. Extensive field data collection and analysis efforts confirmed the accepted values for perception–reaction time and deceleration rate as 1.0 s and 10 ft/s^2, respectively. The data also provided a basis for 85th percentile speed estimations for through and left-turning vehicles. Accounting for start-up delay was also justified. From the findings, a succinct guideline for timing the yellow change and red clearance intervals was formulated on the basis of the ITE kinematic equation and its associated variable values. The recommended guideline encourages a uniform practical application to provide a framework that can be easily adopted into transportation agency policy.

If agreement with and acceptance of the guidelines by the traffic engineering community are assumed, then there does not appear to be any justification for additional research into this issue, specifically, the formulation of the equation and its associated values. However, further research is suggested to evaluate the safety impacts associated with implementing a red clearance interval, as previous studies have yet to conclusively confirm a safety benefit.

REFERENCES

1. *Manual on Uniform Traffic Control Devices.* FHWA, U.S. Department of Transportation, 2009.
2. *Traffic Engineering Handbook,* 6th ed. ITE, Washington, D.C., 2009.
3. *Manual of Traffic Signal Design.* ITE, Washington, D.C., 1982.
4. *Traffic Control Devices Handbook 2001.* ITE, Washington, D.C., 2001.
5. *Signalized Intersections: Informational Guide.* FHWA, U.S. Department of Transportation, 2004.
6. *Traffic Signal Timing Manual.* FHWA, U.S. Department of Transportation, 2009.
7. *Yellow Change Intervals.* Memorandum. FHWA, U.S. Department of Transportation, June 5, 2008. http://safety.fhwa.dot.gov/policy/memo071008#yellow.
8. *Determining Vehicle Change Intervals: A Proposed Recommended Practice.* ITE, Washington, D.C., 1985.
9. McGee, H., Sr., K. Moriarty, T. Gates, K. Eccles, R. Retting, and M. Liu. *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections.* Transportation Research Board of the National Academies, Washington, D.C., in press.
10. Gates, T. J., H. McGee, Sr., K. Moriarty, and H.-U. Maria. Comprehensive Evaluation of Driver Behavior to Establish Parameters for Timing of Yellow Change and Red Clearance Intervals. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2298,* Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 9–21.
11. *Traffic Engineering Handbook.* ITE, Washington, D.C., 1965.
12. Benioff, B., F. C. Dock, and C. Carson. *A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals.* Report FHWA-RD-78-47. FHWA, U.S. Department of Transportation, 1980.
13. *Uniform Vehicle Code.* National Committee on Uniform Traffic Laws and Ordinances, Strasburg, Va., 2000.
14. Parsonson, P. S. Evaluation of Driver Behavior at Signalized Intersections. Discussion. In *Transportation Research Record 904,* TRB, National Research Council, Washington, D.C., 1983, pp. 10–20.
15. *Traffic Engineering Handbook.* ITE, Washington, D.C., 1982.
16. Parsonson, P. S., and A. Santiago. *Design Standards for Timing the Traffic Signal Clearance. Interval Must Be Improved to Avoid Liability.* Compendium of Technical Papers. ITE, Washington, D.C., 1980, pp. 67–71.
17. Fitch, J. W., K. Shafizadeh, W. Zhao, and W. D. Crowl. A Rational Method for Setting All-Red Clearance Intervals. *ITE Journal,* Vol. 81, No. 2, 2011, pp. 16–20.
18. *A Policy on Geometric Design of Highways and Streets,* 5th ed. AASHTO, Washington, D.C., 2004.
19. *Quieter Cars and the Safety of Blind Pedestrians, Phase 2: Development of Specifications for Vehicle Countermeasure Sounds.* Report No. DOT HS 811 496. NHTSA, U.S. Department of Transportation, 2011.
20. Knoblauch, R., M. Nitzburg, R. Dewar, J. Templer, and M. Pietrucha. *Older Pedestrian Characteristics for Use in Highway Design.* Report FHWA-RD-93-177. FHWA, U.S. Department of Transportation, 1995.
21. *Highway Capacity Manual.* TRB, National Research Council, Washington, D.C., 2000.

The Traffic Control Devices Committee peer-reviewed this paper.

Comprehensive Evaluation of Driver Behavior to Establish Parameters for Timing of Yellow Change and Red Clearance Intervals

Timothy J. Gates, Hugh McGee, Sr., Kevin Moriarty, and Honey-Um Maria

In 2009, NCHRP Project 03-95 sought to develop a comprehensive, uniform set of recommended guidelines for determining safe and operationally efficient yellow change and red clearance (i.e., all-red) intervals at signalized intersections. Before guideline development, a comprehensive field study of driver behavior was performed to establish the three primary parameters found in a commonly used kinematic equation (the ITE equation): perception–reaction time, deceleration rate, and approach speed. To provide broad nationwide representation of driver characteristics, site conditions, and traffic signal timing practices, 83 study approaches were evenly selected from four regions of the United States: Southeast Michigan, Central Florida, Southern California, and Greater Washington, D.C. Driver behavior data were collected at each site with an elevated high-definition video camera. The resulting data set included 2,458 first-to-stop dilemma zone vehicles that were less than 5.5 s upstream of the intersection at the start of the yellow interval. The study's overall mean brake response time of 1.00 s was identical to ITE's recommended default value of 1.0 s for timing of the yellow interval, and the overall mean deceleration rate of 10.08 ft/s^2 was very close to ITE's recommended deceleration rate of 10 ft/s^2. The study determined that the 85th percentile speed of free-flowing through-moving vehicles approaching a signalized intersection may be accurately estimated by adding 7 miles per hour (mph) to the posted speed limit on the approach. Similarly, the 85th percentile approach speed for free-flowing left-turning vehicles may be accurately estimated by subtracting 5 mph from the posted speed limit.

For more than 70 years, the subject of yellow and red signal indications has been an important topic for traffic engineering professionals, academics, law enforcement officers, politicians, and the general public. Although there has been much discussion, both recently and historically, calling for nationwide uniformity for timing of the yellow change and red clearance (i.e., all-red) intervals, there currently is no consensus. The lack of a national standard, recommended practice, or even a recognized set of guidelines for determining the duration of yellow change and red clearance intervals, has forced agencies to establish their own practices.

Although no uniform standard or guideline currently exists, the most widely used method for calculating the necessary change and clearance periods in the United States is the following ITE equation (*1*):

$$\mathrm{CP} = t + \frac{V}{2a + 64.4g} + \frac{W + L}{V} \tag{1}$$

where

CP = change period (s),
t = perception–reaction time (typically assumed as 1 s),
V = approach speed (ft/s),
a = deceleration rate (typically assumed as 10 ft/s^2),
g = percentage of grade divided by 100 (positive for upgrade, negative for downgrade),
W = width of intersection (ft), and
L = length of vehicle (ft).

The original form of this equation was first described by Gazis et al. (*2*) in 1960 as a means by which to eliminate the dilemma zone for drivers approaching an intersection at the end of the green signal. Referred to as the "ITE equation," "kinematic equation," or "Gazis' equation," it has served as the ITE proposed recommended practice since 1985 (*3*). Although ITE never formally adopted this proposed recommended practice, it remains the most widely used procedure for the timing of yellow change and red clearance intervals (*4*).

In 2009, NCHRP Project 03-95 sought to develop a comprehensive and uniform set of recommended guidelines for determining safe and operationally efficient change and clearance intervals at signalized intersections (*5*). However, before these guidelines could be developed, it was necessary to understand the behavioral characteristics of drivers approaching an intersection during the yellow and red intervals. Although relevant behavioral characteristics such as perception–reaction times, deceleration rates, and approach speeds have been evaluated in past research, there are limitations to the current body of knowledge. Most of the fundamental driver behavior studies that provided the basis for the timing of change and clearance intervals were performed more than 20 years ago; many of the older studies tend to be limited in scope because they excluded various conditions related to drivers, traffic, intersections, or other site-related phenomena. Furthermore, determination of appropriate approach speed is often still problematic because agencies frequently

H. McGee, Sr., and K. Moriarty, Vanasse Hangen Brustlin, Inc., 8300 Boone Boulevard, Suite 700, Vienna, VA 20165. T. J. Gates and H.-U. Maria, Department of Civil and Environmental Engineering, Wayne State University, 5050 Anthony Wayne Drive, Detroit, MI 48202. Corresponding author: T. J. Gates, tjgates@wayne.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 9–21.
DOI: 10.3141/2298-02

do not have the resources available to collect local speed data, and there is little guidance as to the appropriate methods for collecting these data. Consequently, it was determined that a comprehensive nationwide field evaluation of driver behavior was needed to establish the parameters used for the timing of change and clearance intervals. This evaluation focused on three objectives associated with the primary variables found in the kinematic equation:

- Determine brake response (perception–reaction) times for timing the yellow change interval (t in Equation 1),
- Determine deceleration rates for timing the yellow change interval (a in Equation 1), and
- Determine approach speeds for use in the timing of yellow change and red clearance intervals (V in Equation 1).

FIELD STUDY

A comprehensive field study of driver behavior was performed to satisfy the aforementioned research objectives. It was initially necessary to compile a comprehensive list of site-related factors that potentially affect driver behavior when the driver approaches an intersection during the yellow interval. These factors provided the basis for selection of the study sites; they were developed on the basis of expert opinion along with the results of the literature review, particularly the criteria suggested by Bonneson et al. (*6*). These factors included

- Speed limit,
- Area type,
- Intersection clearing width,
- Cycle length,
- Duration of the yellow change interval,
- Duration of the red clearance interval,
- Proximity to the upstream signal,
- Type of opposing left-turn signalization, and
- Approach grade.

Site Selection

Eighty-three signalized intersection approaches were selected for use in this field study. The study approaches were selected to collectively provide representative balance within each of the categories listed above and were evenly distributed among four distinct regions of the United States: Southeast Michigan (Ann Arbor and metropolitan Detroit), Central Florida (Orlando and The Villages area), Southern California (Los Angeles and Orange County), and Metropolitan Washington, D.C. (Northern Virginia and Maryland). These regions were selected to provide a broad nationwide representation of driver characteristics, site conditions, and traffic signal timing practices while also considering proximity and familiarity of the research team with the particular area.

The research team initially identified potential intersection approaches on the basis of its familiarity with the locations or with assistance from personnel within the local transportation agencies. The team also relied on satellite and street-level imagery that was freely available on the Internet to assist in site selection. No more than two approaches from each intersection were used in the study. Final selection of the specific study sites was based on the following criteria:

- Adequate representation within the various site factors;
- Adequacy of the site for the setup of video camera equipment;
- Relatively straight, right-angle approaches; and
- Two through lanes on the approach separated by visible lane-line markings.

The primary characteristics of the study sites are summarized in Table 1. A full factorial study design was neither practical nor feasible.

Data Collection Procedures

Between October 2009 and July 2010, driver behavioral data were collected at the 83 signalized intersection approaches. Vehicles

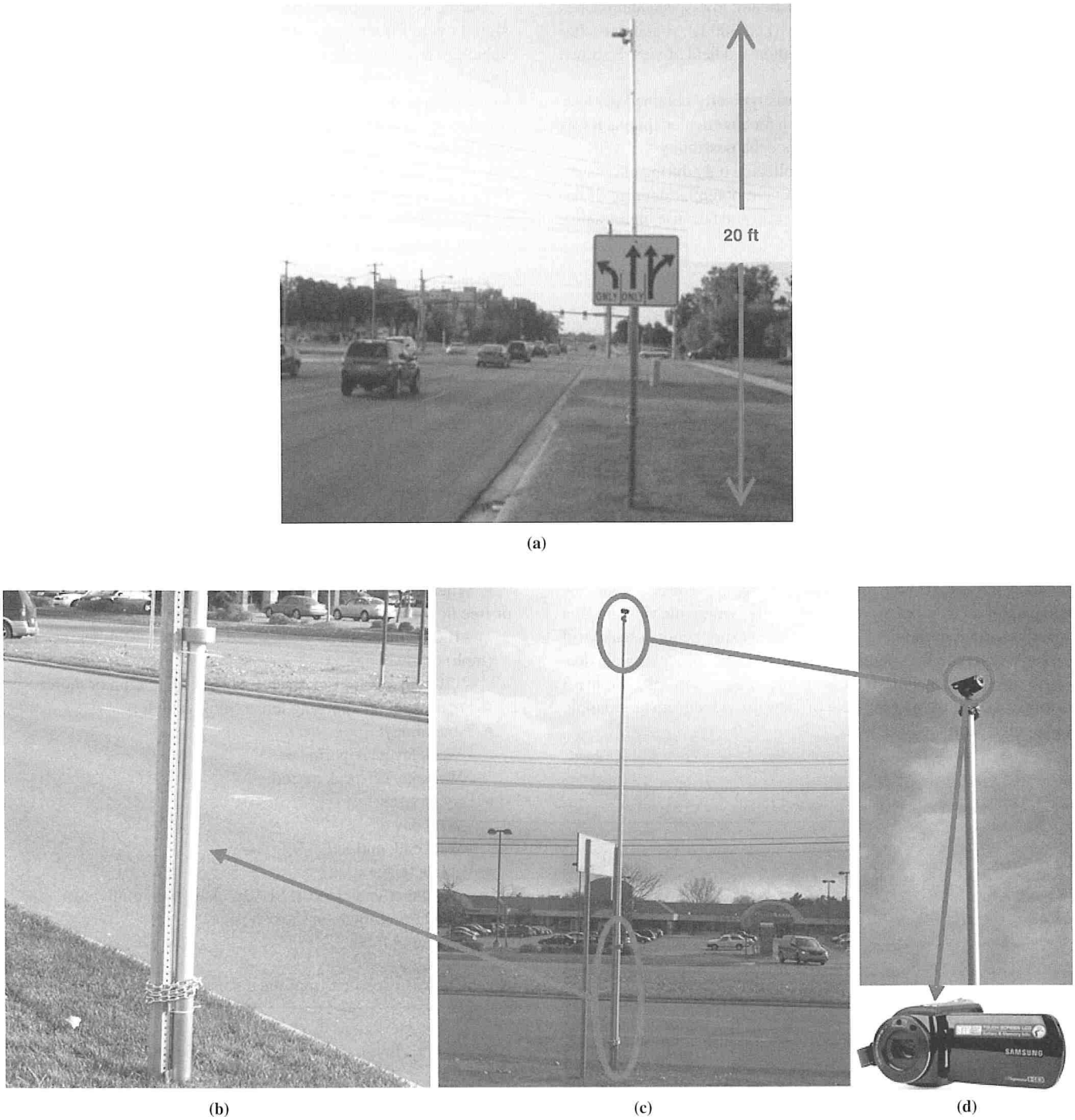
TABLE 1 Summary of Site Characteristics, by Factor

Factor	Category	Number of Sites	Percentage of All Sites
State	Michigan	23	28
	Florida	20	24
	California	21	25
	Virginia	11	13
	Maryland	8	10
Speed limit (mph)	≤40 (low speed)	30	36
	45	36	43
	≥50 (high speed)	17	20
Area type	Urban (downtown)	6	7
	Suburban	70	84
	Rural (outside of incorporated boundaries)	7	8
Clearing width (ft) (from stop line to far curb)	≤48	3	4
	48 to 72	8	10
	72 to 96	22	27
	96 to 120	17	20
	>120	33	40
Proximity to upstream signal	No upstream signal within 0.5 mi	21	25
	Upstream signal within 0.5 mi	62	75
Cycle length (s)	<90	22	27
	90 to 120	13	16
	120 to 180	35	42
	>180	13	16
Duration of yellow change interval (s)	≤4.0	26	31
	4.1 to 4.5	29	35
	4.6 to 5.0	21	25
	≥5.1	7	8
Duration of red clearance interval (s)	None	9	11
	<1.0	29	35
	1.1 to 2.0	33	40
	2.1 to 3.0	10	12
	>3.0	2	2
Opposing left-turn signalization	Protected only	58	70
	Permissive only	4	5
	Protected–permissive	7	8
	Permissive–protected	4	5
	Prohibited or none	10	12
Grade[a]	Downgrade	2	2
	Level	78	94
	Upgrade	3	4

[a]Downgrade = greater than −3%; level = between −3% and 3%; and upgrade = greater than 3%.

approaching an intersection were covertly recorded with a high-definition video camera mounted on a modular 20-ft-tall aluminum pole that was securely strapped and locked to a stable roadside signpost or utility pole. An example of the video camera setup is shown in Figure 1.

The camera was installed at the particular subject approach and was aimed downstream toward the intersection to provide a field of view ranging from 300 to 600 ft along the approach, including the intersection and the traffic signal in the field of view. This vantage point allowed for observation of all characteristics relevant to the behavioral evaluation, including the traffic signal indication, brake light indications, location of the vehicle with respect to the stop line, vehicle type, spacing between successive vehicles, whether the vehicle stopped or went through the intersection, and whether red light running occurred. The upstream edge of the camera viewing area was positioned approximately 100 ft beyond the upstream edge of the dilemma zone, which was computed with the kinematic equation on the basis of the speed limit, existing yellow duration, and default parameters for all other variables. Thus, high-speed approaches were provided with the greatest field of view (i.e., from the intersection to 600 ft upstream), as



(a) (b) (c) (d)

FIGURE 1 Typical video camera setup for recording of driver behavior data.

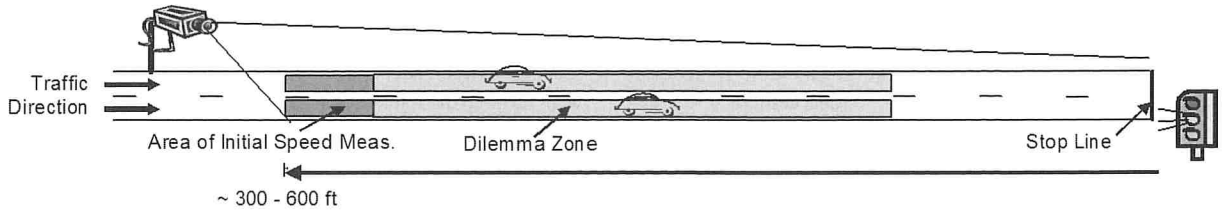


FIGURE 2 Typical video camera field of view (Meas. = measurement).

the dilemma zone occurred farther upstream; low-speed approaches required a much shorter field of view (i.e., 300 ft). A general schematic of a typical video camera installation and field of view is shown in Figure 2.

Approximately 3 to 5 h of video was typically obtained per site. With few exceptions, all data collection for a particular approach was performed within a single day, to reduce the possibility that vehicles would be counted twice. Data were collected only during dry conditions and during daylight hours to prevent potential damage of the video camera during periods of heavy rain and because high-quality video could not be recorded at night.

Data Extraction

The videos were manually reviewed on a computer with QuickTime software. The relevant behavioral data were extracted for vehicles traveling on the subject approach during each change and clearance interval. QuickTime allowed for frame-by-frame review of the videos to determine the relevant vehicular location and time information. The video was recorded at a rate of 60 frames per second, which allowed time to be recorded to the nearest 0.0167 s, as displayed in the video player. The regular pattern of the white broken lane-line pavement markings (e.g., 10-ft marking with a 30-ft gap) provided a convenient field reference marker for determining the location of a vehicle with respect to the stop line. The actual lengths and gaps of all lane-line pavement markings within the viewing area were measured in the field. With these measurements, a scale grid was overlaid onto the computer screen; this grid allowed a vehicle's approximate distance from the stop line to be determined. Figure 3 displays a screenshot of the field of view provided from an example of an intersection video frame.

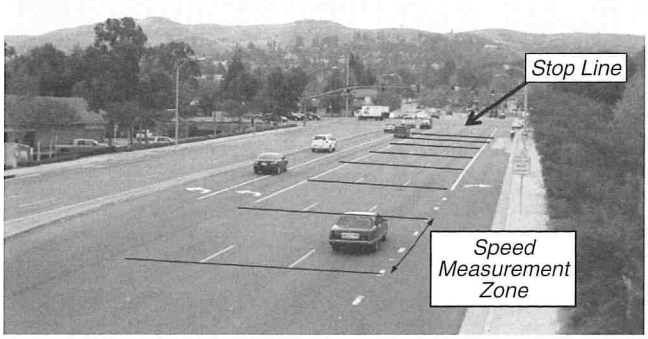


FIGURE 3 Example of screenshot from intersection video with simulated grid.

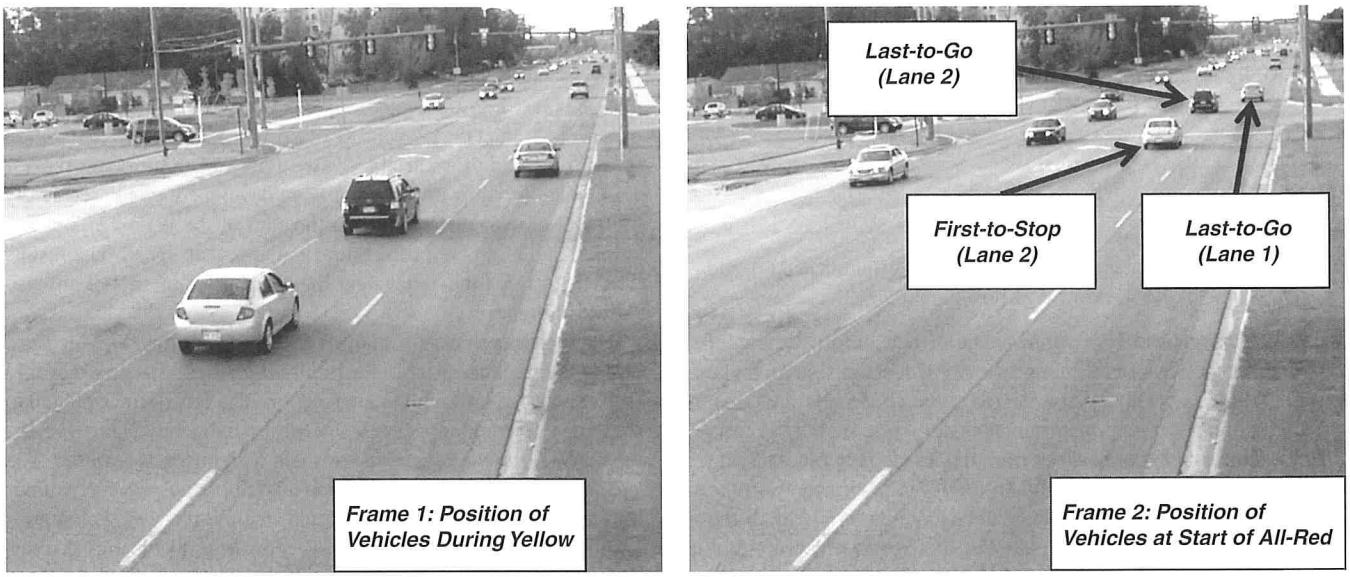
During the video review process, data were obtained during each signal cycle for each nonturning vehicle that was either the last vehicle to go through the intersection or the first vehicle to stop in each lane along the subject approach. An example depicting the designation of last-to-go and first-to-stop vehicles is shown in Figure 4. Turning vehicles were excluded because turning drivers must typically decelerate to complete the turning maneuver. That is, brake response times and deceleration rates for turning vehicles are not necessarily indicative of a response to the yellow indication.

The following information was obtained from the video for each subject vehicle:

- Time to traverse the initial speed measurement zone;
- Time and location at the start of the yellow signal;
- Action of the vehicle:
  - Stopped,
  - Went through and crossed the stop line before the end of the yellow signal, or
  - Went through and crossed the stop after the end of the yellow signal (i.e., red light running);
- Time and location at the start of brake light illumination (stopping vehicles only);
- Time that the vehicle stopped (stopping vehicles only);
- Intersection entry time after the start of the red signal (red light running vehicles only);
- Whether the vehicle was a platoon leader, platoon follower, or free flowing, according to the following spacing requirements:
  - 100 ft at locations with speed limits of 40 miles per hour (mph) or lower and
  - 150 ft at locations with speed limits of 45 mph or more;
- Presence of an opposing left-turning vehicle;
- Time of day:
  - Weekday peak period or
  - Weekday off-peak period;
- Day of week:
  - Weekday or
  - Weekend; and
- Vehicle type:
  - Passenger vehicle (car, SUV, pickup, van, minivan);
  - Single-unit truck, bus, or RV; or
  - Multiunit truck.

A vehicle was excluded from the data set for any of the following reasons:

- Was not a first-to-stop or last-to-go vehicle within the respective lane,
- Turned right or left or made a U-turn at the intersection,
- Began braking before the onset of the yellow signal (stopping vehicles only),



(a) (b)

FIGURE 4 Example of designation of last-to-go and first-to-stop vehicles.

- Turned out of a driveway within the camera's view,
- Approached at an unusually low rate of speed (<15 mph),
- Exhibited unusual or erratic behavior, or
- Encountered a queue on the subject approach.

The video frame number, which was later converted to a time-based measurement by use of the recorded frame rate of 60 frames per second, was used to determine all time-related information. Brake light indications and traffic signal indications were defined on the basis of the first video frame in which illumination of any of the respective displays could be visibly detected. Vehicle location estimates were made with respect to the front wheels. The reviewers who assessed the stop times of braking vehicles were trained to pause the video immediately before full cessation of motion to account for the slight perception lag that would occur upon identification of the vehicle's true stop time. This time was recorded on the data collection form, and the video was immediately advanced to determine if the vehicle had truly stopped or if any additional forward motion occurred, in which case the measurement was discarded. The convergence of three independent trials that were each within 10 frames (one-sixth of 1 s) of each other was used to determine the time that the vehicle stopped. The average of the three trials was recorded as the vehicle's stop time. Occasionally, certain vehicular data, such as brake light indications, location information, or the time that the vehicle stopped, could not be obtained for a given subject vehicle; in such cases, the record for the vehicle was incomplete. The inability to obtain a complete data record was caused by a variety of factors, but most commonly occurred as a result of obstructions from other approaching vehicles or from sun glare.

Red light running events were defined as cases in which the front of the vehicle reached the stop line after the onset of the red indication. This definition follows the so-called permissive yellow rule and is consistent with the *Uniform Vehicle Code* (*7*). Demographic information of individual drivers was not obtained during the field study, as the video data collection strategy did not allow for the collection of information such as the age, sex, experience, and route familiarity of each individual driver.

Data Reduction

More than 328 h of video were obtained—an average of nearly 4 h for each of the 83 sites. The raw time and vehicle location information obtained from the videos was used to compute approach speeds, estimated travel time to the intersection, brake response (perception–reaction) times, and deceleration rates. Approach speeds before the start of the yellow signal were computed with the following equation:

$$S = \frac{d_S}{t_S} \tag{2}$$

where

S = approach speed before the start of the yellow signal (ft/s),
d_S = speed measurement distance (ft), and
t_S = time to traverse the speed measurement distance (s).

The travel time to the stop line at the onset of the yellow signal was estimated by dividing the subject vehicle's distance from the stop line at the onset of the yellow signal by its approach speed. This method provided only a prediction of the travel time on the basis of the approach speed and distance upstream and was not the actual travel time, which could differ from the predicted value as a result of acceleration or deceleration. The following equation was used to calculate the predicted travel time to the stop line at the start of the yellow signal:

$$\mathrm{TT} = \frac{d_Y}{S} \tag{3}$$

where TT is the predicted travel time to the stop line at the start of the yellow signal (s) and d_Y is the distance from the stop line at the start of the yellow signal (ft).

Brake response times were computed as the absolute difference between the time at the start of the yellow signal and the time when the brake lights became visibly illuminated:

$$\mathrm{BRT} = t_{\mathrm{BL}} - t_Y \tag{4}$$

where

BRT = brake response time (s),
t_{BL} = time when brake lights became visibly illuminated (s), and
t_Y = time at the start of the yellow signal (s).

Brake response time is a common field-measured estimate of perception–reaction time. Although the two values are similar, brake response time is typically considered to be slightly longer than perception–reaction time, as it includes any lag time between the driver's removing his or her foot from the accelerator and applying the brake (i.e., coasting), which could not be quantified from the videos. It also was not possible to determine cases in which the driver prematurely removed his or her foot from the accelerator before the start of yellow signal but did not apply the brakes until after the yellow signal.

The approach speed and braking time were used to compute the average deceleration rate for each vehicle. Although it is acknowledged that drivers may use variable deceleration rates during braking (i.e., jerking), it was not practical to measure more than one deceleration rate for each subject vehicle. Because the speed measurements were taken immediately before the onset of the yellow signal, any impact on speed caused by coasting before braking was assumed to be negligible. Braking time was computed as the absolute difference between the time that the brake lights became visibly illuminated and the time that the vehicle stopped. The following formula was used to compute the average deceleration rate:

$$D_{\mathrm{avg}} = \frac{S}{t_{\mathrm{stop}} - t_{\mathrm{BL}}} \tag{5}$$

where D_{avg} is the average deceleration rate (ft/s^2) and t_{stop} is the time that the vehicle stopped (s).

Establishment of Dilemma Zone Boundaries

The vehicular observation data were tabulated, organized, and coded into a single data file for detailed analysis. The additional independent variables were appropriately coded for each of the subject vehicles. The initial data set included 7,482 vehicle records collected from the 83 sites. The data set was then further stratified to include a more concise range of vehicular arrivals to more accurately represent the behavior of drivers in the dilemma zone. The dilemma zone (more accurately termed the "decision zone" for this study) has been defined in the literature as the area upstream of the intersection between which 10% and 90% of drivers will stop in response to the yellow indication (*8*). Previous research has shown that the 10% to 90% stopping region generally corresponds to drivers who are between 2.5 s and 5.5 s upstream of the intersection at the start of the yellow signal (*9*).

Initial investigation of the full data set showed good agreement with the 2.5- to 5.5-s dilemma (decision) zone region: 7.9% of drivers stopped when approximately 2.5 s upstream at the start of the yellow signal, and 93.1% of drivers stopped when approximately 5.5 s upstream. These values are reflected in Figure 5, which displays the percentage of vehicles stopping or going through the intersection versus the travel time to the stop line at the start of the yellow signal.

To provide further confirmation of the dilemma zone boundaries, a basic binary logistic regression model was developed from the full data set to predict the probability of a driver's stopping as a function of a set of predictor variables. Two forms of the logistic regression model for stop–go behavior were developed: one based solely on travel time to the stop line at the start of the yellow signal and another including both travel time and approach speed. The results from the travel time–dependent logistic regression model are also shown in Figure 5.

The logistic regression analysis found travel time to be the most highly significant variable for predicting whether a driver would stop or proceed through the intersection. The travel times related to stopping probabilities of .1 and .9 were approximately 2.5 s and 5.2 s, respectively, providing good agreement with previous research. The speed-dependent logistic regression model showed relatively minor differences in the upstream and downstream travel time boundaries with respect to approach speed. As a result, uniform travel times were selected to represent the upstream and downstream dilemma zone boundaries for all approach speeds. Thus, to provide an accurate representation of driver behavior in the dilemma zone at all approach speeds and to be consistent with previous research, the data set was filtered to include only

- Last-to-go vehicles (including red light runners) that were more than 2.5 s upstream of the intersection at the start of the yellow signal and
- First-to-stop vehicles that were less than 5.5 s upstream of the intersection at the start of the yellow signal.

The finalized data set based on the dilemma zone boundaries of 2.5 and 5.5 s included 4,572 vehicles that were classified as follows: 2,458 stopping vehicles (53.8%); 1,999 vehicles that went through, crossing the stop line before the red signal (43.7%); and 115 red light running vehicles (2.5%).

ANALYSIS AND RESULTS

Brake Response Time and Deceleration Rate

The data set included 2,458 first-to-stop dilemma zone vehicles that were less than 5.5 s upstream of the intersection at the start of the yellow interval. The overall descriptive statistics for brake response time and deceleration rate for these vehicles were as follows:

- Brake response time:
  - Mean = 1.00 s,
  - Standard deviation = 0.37 s,
  - 15th percentile = 0.68 s, and
  - 85th percentile = 1.33 s and
- Deceleration rate:
  - Mean = 10.08 ft/s^2,
  - Standard deviation = 2.83 ft/s^2,
  - 15th percentile = 7.32 ft/s^2, and
  - 85th percentile = 12.89 ft/s^2.

Figure 6 displays the cumulative distributions of brake response times and deceleration rates for the first-to-stop vehicles observed in this study. Also included in the figures, for comparative pur-

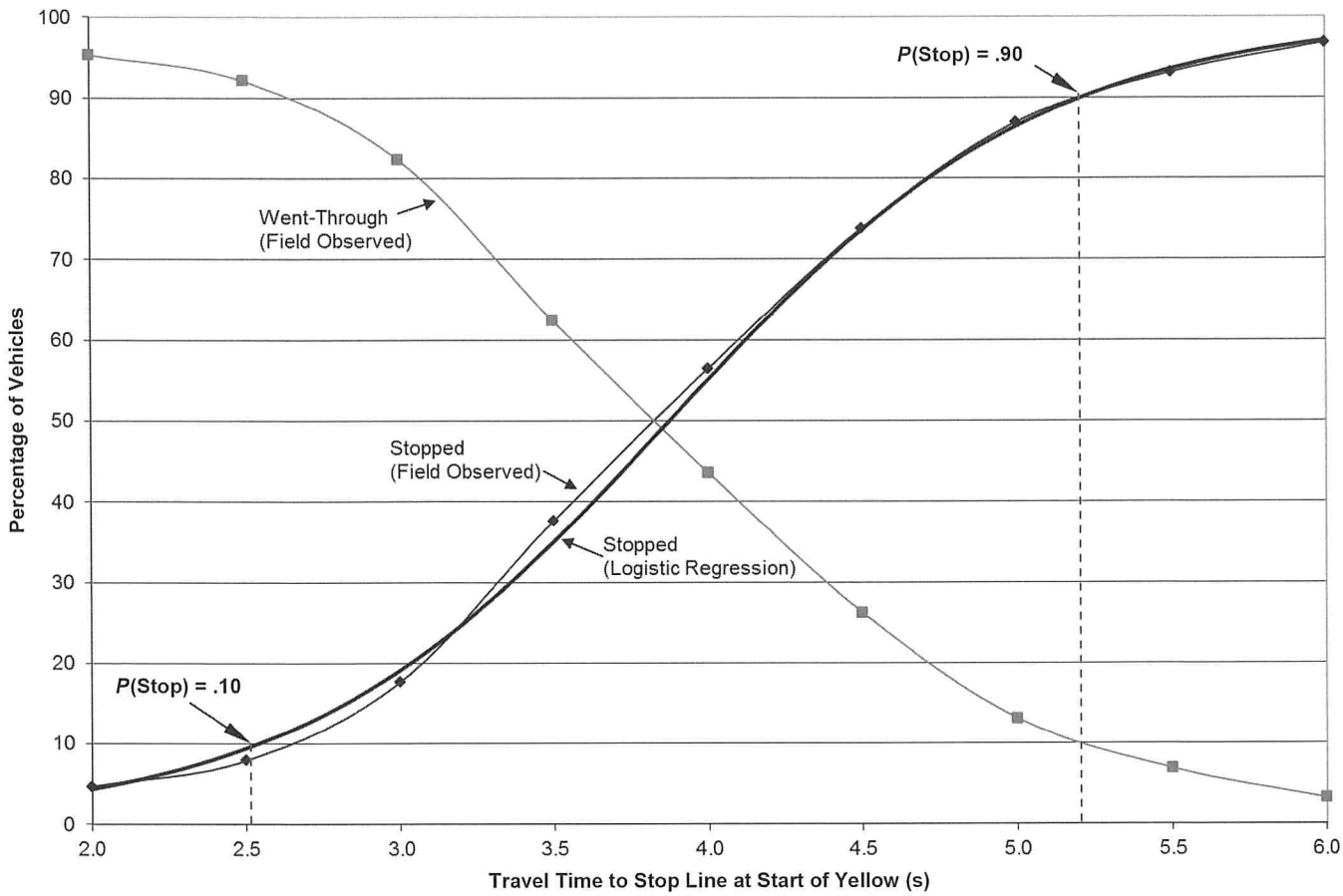


FIGURE 5 Percentage of vehicles stopping or going through intersection versus travel time to stop line at start of yellow signal.

poses, are the distributions of data reported in select previous studies (*10–13*).

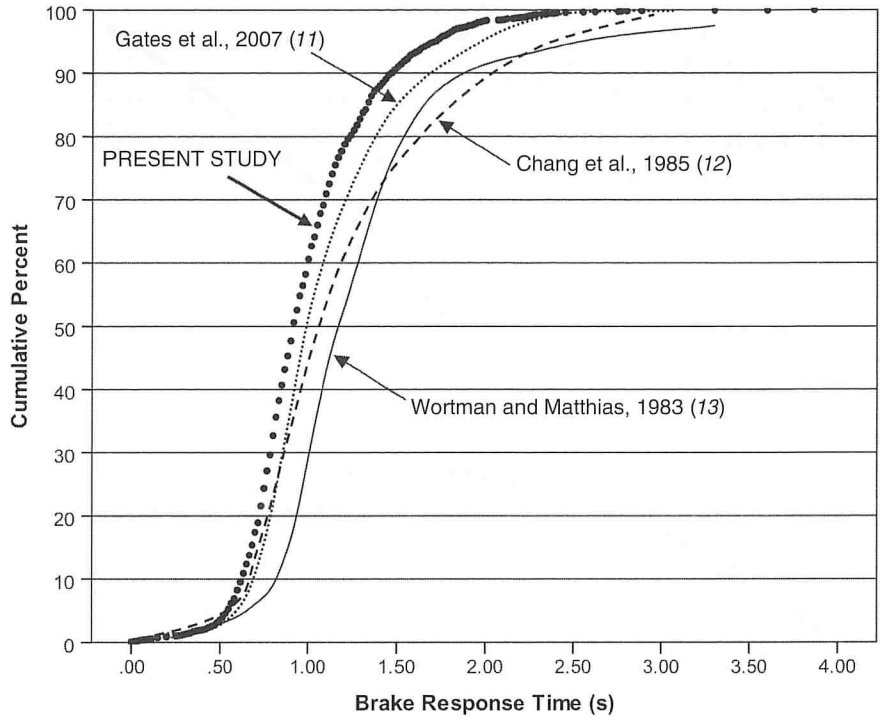
As shown in Figure 6*a*, the distributions of brake response times observed in this study were in good agreement with, albeit slightly shorter than, data observed in previous studies (*11–13*). The observed mean brake response time of 1.00 s was identical to the default perception–reaction time value of 1.0 s recommended by ITE for timing of the yellow interval (*1, 3*).

The deceleration rates observed in this study were also similar to values observed in previous research. Figure 6*b* shows a distribution of deceleration rates for the data observed in this study that is nearly identical to that from a 2007 Wisconsin study by Gates et al. (*11*). The 50th percentile deceleration rate of 9.65 ft/s^2 observed in the present study was very similar to the 50th percentile values found in each of the previous studies (*10–13*), with the exception of the 1983 study by Wortman and Matthias (*13*). The overall mean deceleration rate of 10.08 ft/s^2 is very close to ITE's recommended deceleration rate of 10 ft/s^2 (*1, 3*).

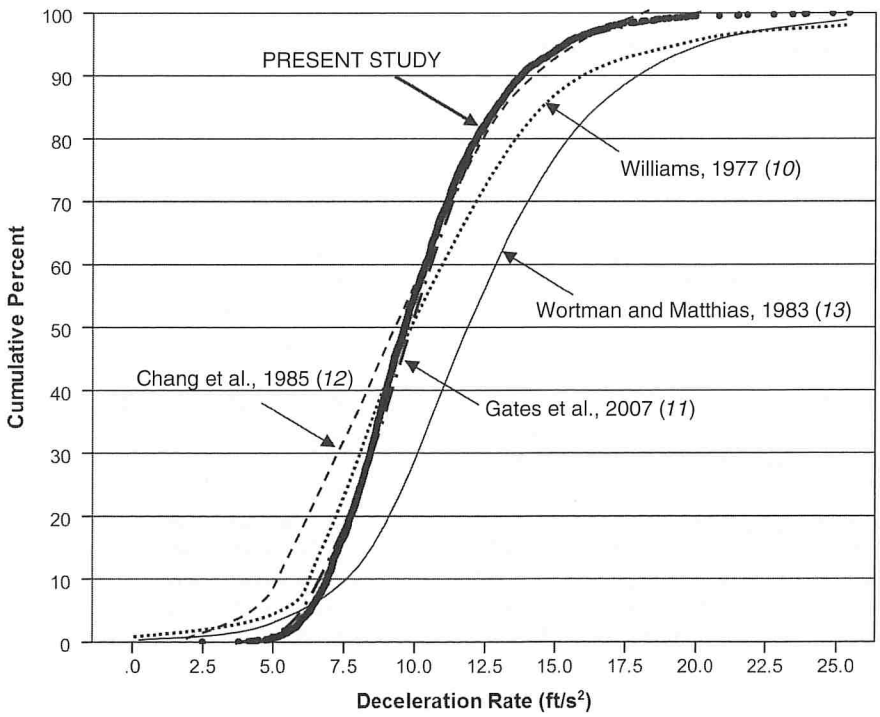
Separate univariate analyses of variance (ANOVAs) in SPSS Version 18 were used to analyze the data (*14*). The independent variables included both those related to the characteristics of the location and the sampling variables. The dependent variable was either brake response time or deceleration rate.

The ANOVA found several variables that significantly affected brake response time, deceleration rate, or both. All statistical inference was performed at the 95% confidence level. The statistical significance of the study variables is summarized in the following list, in order of the relative magnitude of the *F*-statistic:

- Factors that were statistically significant:
  - Brake response time:
    1. Travel time to stop line at the start of the yellow signal,
    2. Approach speed,
    3. Deceleration rate,
    4. Approach grade, and
    5. Speed limit;
  - Deceleration rate:
    1. Travel time to stop line at the start of the yellow signal,
    2. Approach speed,
    3. Brake response time,
    4. Approach grade,
    5. Speed limit, and
    6. Duration of the yellow interval; and
- Factors that were not statistically significant:
  - Brake response time:
    1. Vehicle type,
    2. Peak versus off-peak period,
    3. Weekday versus weekend,
    4. Signal proximity,
    5. Intersection clearing width,
    6. Presence of opposing left-turner,
    7. Platoon versus free-flowing vehicles,



(a)



(b)

FIGURE 6 Cumulative distributions of brake response times and deceleration rates for first-to-stop vehicles: (*a*) brake response times and (*b*) deceleration rates.

8. Cycle length,
9. Duration of the yellow interval, and
10. Duration of the red clearance interval and

– Deceleration rate:
1. Vehicle type,
2. Peak versus off-peak period,
3. Weekday versus weekend,
4. Signal proximity,
5. Intersection clearing width,
6. Presence of opposing left-turner,
7. Platoon versus free-flowing vehicles,
8. Cycle length, and
9. Duration of the red clearance interval.

The most highly significant factors were those related to the vehicle's position at the start of the yellow signal and to the behavior of the driver. Most of the site-related characteristics or temporal characteristics were not significant. Table 2 provides the descriptive statistics for each categorical level of the statistically significant factors.

Further investigation of the descriptive statistics shown in Table 2 along with the parameter estimates from the ANOVA model revealed the following information about the statistically significant variables:

- Brake response times:
  – Decreased as approach speed increased,
  – Increased as travel time to the intersection at the start of the yellow signal increased,
  – Increased as the deceleration rate increased,
  – Decreased for steep downgrades, and
  – Decreased as speed limit increased and
- Deceleration rates:
  – Increased as approach speed increased,
  – Decreased as travel time to the intersection at the start of the yellow signal increased,
  – Increased as the brake response time increased,
  – Increased for steep upgrades and decreased for steep downgrades,
  – Increased as speed limit increased, and
  – Increased as the duration of the yellow interval increased.

Approach Speeds of Through Vehicles

Approach speeds were computed for 3,632 free-flowing vehicles that were randomly sampled from 60 of the study sites with posted speed limits ranging from 25 to 55 mph. At least four sites were included within each speed limit category. The video-based speed measurement procedure described previously was used to determine the speeds at the upstream portion of the dilemma zone. Both through and stopping vehicles were sampled, as long as the brakes were not applied either before or during the speed measurement. Turning vehicles were excluded.

The sites were then grouped together on the basis of speed limit. Three different methods were used to calculate the summary statistics for each speed limit category:

1. The mean and 85th percentile of all speeds pooled within the respective speed limit category,
2. Average of the mean and 85th percentile speed values of each location within the respective speed limit category, and
3. Least squares regression (simple linear) for the mean and 85th percentile speed values obtained for each site versus the speed limit at the site.

The simple least squares linear regression model showed very good fit for both the mean and 85th percentile speed data plotted versus the speed limit for each of the 60 sites. The regression equations for prediction of the mean speed (S_{mean}) and 85th percentile approach

TABLE 2 Descriptive Statistics for Each Statistically Significant Factor

			Brake Response Time (s)				Deceleration Rate (ft/s^2)			
Factor	Category	Number of Vehicles	Mean	SD	15th Percentile	85th Percentile	Mean	SD	15th Percentile	85th Percentile
Approach speed (mph)	20 to 30	263	1.07	0.38	0.77	1.42	8.13	2.52	5.89	10.68
	30 to 40	763	1.03	0.39	0.72	1.38	9.26	2.43	7.02	11.36
	40 to 50	1,022	0.97	0.36	0.67	1.30	10.46	2.64	7.91	13.06
	50 to 60	353	0.94	0.35	0.63	1.25	11.90	2.67	9.53	14.50
Travel time to stop line (s)	<4.5	1,225	0.93	0.33	0.65	1.22	11.28	2.94	8.50	14.10
	≥4.5	1,233	1.06	0.40	0.73	1.42	8.90	2.14	6.85	10.93
Grade[a]	Downgrade	44	0.66	0.42	0.21	1.21	9.41	2.00	7.43	12.13
	Level	93	1.03	0.42	0.70	1.50	10.33	2.66	7.98	12.60
	Upgrade	127	1.05	0.34	0.75	1.40	12.02	3.22	8.56	15.47
Speed limit (mph)	25 to 30	267					8.21	2.24	6.17	10.67
	35 to 40	540					9.79	2.80	7.32	12.31
	45 to 50	1,444					10.29	2.78	7.63	13.10
	55 to 60	207					11.80	2.53	9.5	14.18
Duration of yellow (s)	≤ 4	591					9.21	2.69	6.75	11.70
	4.1–4.9	1,082					10.06	2.76	7.36	12.83
	≥ 5	785					10.77	2.86	8.11	13.49
	Overall	2,458	1.00	0.37	0.68	1.33	10.08	2.83	7.32	12.89

NOTE: SD = standard deviation; shading = variable not significant.
[a]Downgrade = greater than –3%; level = between –3% and 3%; and upgrade = greater than 3%.

TABLE 3 Mean and 85th Percentile Approach Speeds of Through Vehicles Versus Posted Speed Limit

			Mean Speeds (mph)			85th Percentile Speeds (mph)		
Posted Speed Limit (mph)	Number of Sites	Number of Vehicles Sampled	Pooled	Site Average	Regression Prediction	Pooled	Site Average	Regression Prediction
25	4	368	30.15	30.69	28.40	35.00	35.61	33.48
30	4	325	32.32	32.97	32.53	38.07	37.95	37.91
35	4	320	37.94	37.52	36.66	42.00	41.71	42.33
40	16	719	39.46	39.77	40.79	46.49	46.51	46.75
45	17	893	44.12	43.79	44.92	51.14	49.86	51.18
50	10	644	49.88	50.13	49.06	57.65	57.06	55.60
55	5	363	56.89	55.34	53.19	62.94	61.14	60.02

speed (S_{85th}) as a function of the speed limit on the subject approach were as follows:

$$S_{mean} = 0.8262 \times \text{speed limit} + 7.745 \qquad R^2 = .786 \qquad (6)$$

$$S_{85th} = 0.8846 \times \text{speed limit} + 11.369 \qquad R^2 = .827 \qquad (7)$$

where R^2 is the coefficient of determination of a linear regression.

Table 3 and Figure 7 display the mean and 85th percentile speeds versus the posted speed limit computed by use of each of the three calculation methods. The results showed that mean speeds typically exceeded the speed limit at locations with speed limits of 35 mph or lower and were approximately equal to the speed limit at speed limits between 40 and 55 mph. This finding was not unexpected: speed limits in urban areas are often set lower than the design speed would indicate, often as a result of statutory regulations, thereby prompting a higher level of noncompliance with the speed limit.

Table 4 displays the difference between both the mean and 85th percentile speeds and the speed limit for each of the three calculation methods. When the speed data were pooled together within the respective speed limit category, the overall mean speed was 0.84 mph greater than the speed limit and the overall 85th percentile speed was 7.45 mph greater than the speed limit. When the speed statistics were computed on a site-by-site basis, the mean speeds were, on average, 0.39 mph greater than the speed limit and the 85th percentile speeds were, on average, 6.49 mph greater than the speed limit. The regression model also showed similar results.

The results shown in Table 4 indicate that, for practical purposes, the posted speed limit provides a good estimate of the mean speed of free-flowing vehicles approaching a signalized intersection, particu-

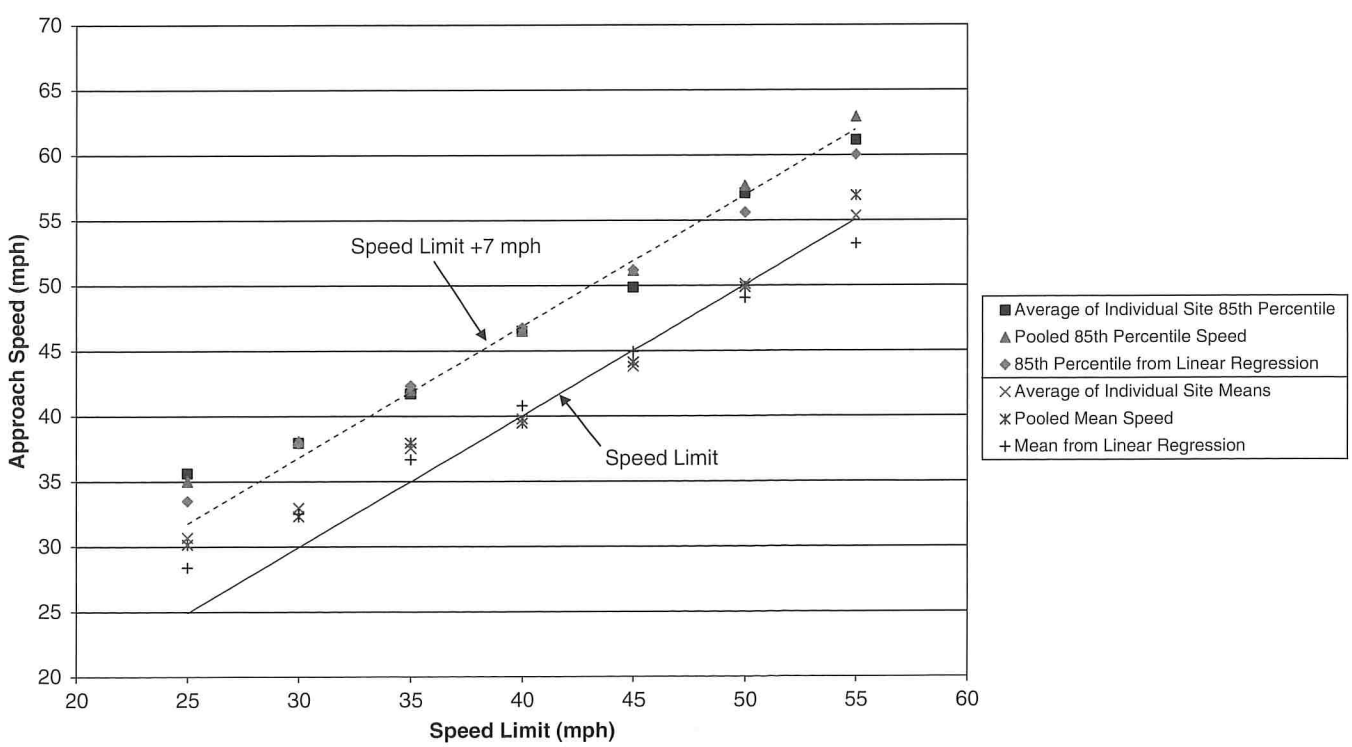


FIGURE 7 Mean and 85th percentile speed versus speed limit.

TABLE 4 Difference Between Mean and 85th Percentile Approach Speeds of Through Vehicles and the Posted Speed Limit

	Mean Speed Minus Speed Limit (mph)			85th Percentile Speed Minus Speed Limit (mph)		
Posted Speed Limit (mph)	Pooled	Site Average	Regression Prediction	Pooled	Site Average	Regression Prediction
25	5.15	5.69	3.40	10.00	10.61	8.48
30	2.32	2.97	2.53	8.07	7.95	7.91
35	2.94	2.52	1.66	7.00	6.71	7.33
40	−0.54	−0.23	0.79	6.49	6.51	6.75
45	−0.88	−1.21	−0.08	6.14	4.86	6.18
50	−0.12	0.13	−0.94	7.65	7.06	5.60
55	1.89	0.34	−1.81	7.94	6.14	5.02
Overall	0.84	0.39	0.79	7.45	6.49	6.75

larly for locations with speed limits of 40 mph or more. However, the speed limit itself typically does not provide an accurate estimate of the 85th percentile speed. The 85th percentile speed was found to exceed the speed limit at all but one of the 60 locations. The overall difference between the 85th percentile speed and the posted speed limit was between 6.5 mph and 7.5 mph, depending on the method used to determine the 85th percentile speed. This difference was more pronounced for roadways with a posted speed limit of 25 mph, as the 85th percentile speed exceeded the speed limit by 8.5 to 10.6 mph, depending on the calculation method used.

Approach Speeds of Left-Turning Vehicles

Approach speeds for free-flowing left-turning vehicles were also measured at 19 of the study sites. The speed limits at the sites ranged from 40 to 55 mph, and each of the five states (four regions) was represented. Locations with speed limits of 35 mph or more were not included, as the operating speed for left-turning vehicles on low-speed approaches was assumed to be the same (or nearly so) as that for through vehicles.

The approach speeds were measured immediately before the yellow signal in the same manner and at approximately the same upstream location as through-moving vehicles. The subject vehicles were typically positioned in the leftmost through lane during the speed measurement and generally merged into the left-turn lane shortly after the speed measurement was made. Speeds were also obtained for vehicles already positioned in the left-turn lane and for vehicles that were braking before or during speed measurement.

The results of the data collection for left-turn approach speed are shown in Table 5. The results show that the overall mean approach speed for left-turning vehicles was 10.59 mph less than the posted speed limit. The overall 85th percentile left-turn approach speed was 4.94 mph less than the posted speed limit. These values reinforced the assumption that drivers typically prepare to decelerate to complete the left-turn maneuver.

Speeds were not measured during the completion of the left-turn maneuver, as these speeds are limited by the geometry of the turning movement. However, with AASHTO's equation for calculating the design speed of horizontal curves and the appropriate design side friction factors (*15*), the average left-turn design speed for the study sites was calculated as 16.3 mph with an 85th percentile value of 18.5 mph. These design speed values are considered conservative on the basis of the design side friction factors provided by AASHTO.

SUMMARY AND CONCLUSIONS

A comprehensive nationwide field study of driver behavior was performed to establish the three primary parameters used in the ITE equation for the timing of yellow change and red clearance intervals: perception–reaction time, deceleration rate, and approach speed. An elevated video camera was used to collect driver behavioral data

TABLE 5 Mean and 85th Percentile Approach Speeds of Left-Turning Vehicles Versus Posted Speed Limit

Posted Speed Limit (mph)	Number of Sites	Number of Vehicles Sampled	Mean Speed (mph)	Mean Speed Minus Speed Limit (mph)	85th Percentile Speed (mph)	85th Percentile Speed Minus Speed Limit (mph)
40	3	90	31.85	−8.15	37.49	−2.51
45	11	332	34.64	−10.36	39.91	−5.09
50	3	88	36.61	−13.39	44.02	−5.98
55	2	60	43.61	−11.39	50.90	−4.10
Overall	19	570	na	−10.59	na	−4.94

NOTE: na = not applicable.

at 83 signalized intersection approaches. The study locations were evenly distributed among four distinct regions of the United States: Southeast Michigan, Central Florida, Southern California, and Greater Washington, D.C. (which encompasses Northern Virginia and Maryland). Data were obtained for 2,458 first-to-stop dilemma zone vehicles that were less than 5.5 s upstream of the intersection at the start of the yellow interval. Analysis of brake response times, deceleration rates, and approach speeds allowed for the following conclusions to be formulated.

Brake Response Times

- The brake response times observed in this study are in good agreement with, albeit slightly shorter than, those in previous research (*11–13*).
- The mean brake response time of 1.00 s observed in this study is equal to the default perception–reaction time value of 1.0 s recommended by ITE for timing of the yellow interval (*1, 3*).
- Drivers approaching at higher speeds have shorter brake response times.
- When they are farther away from the intersection at the start of the yellow signal, drivers have longer brake response times.
- Drivers traveling on steep downgrades have shorter brake response times.

Deceleration Rates

- The deceleration rates observed in this study are very similar to those observed in previous research (*10–13*).
- The overall mean deceleration rate of 10.08 ft/s^2 is nearly equal to ITE's recommended deceleration rate of 10 ft/s^2 (*1, 3*).
- Drivers approaching at higher speeds have higher deceleration rates.
- When they are farther away from the intersection at the start of the yellow signal, drivers have lower deceleration rates.
- Drivers traveling on steep downgrades decelerate at a lower rate.
- Drivers traveling on steep upgrades decelerate at a higher rate.
- Drivers' selection of deceleration rate is dependent on brake response time. If it is assumed that all other factors are held constant, then slower-reacting drivers compensate with greater deceleration rates to stop, while quicker-reacting drivers are able to stop by using lower deceleration rates.

Approach Speeds of Through Vehicles

- The posted speed limit provides an accurate estimate of the mean speed of free-flowing through vehicles approaching a signalized intersection, but it greatly underestimates the 85th percentile speed, which is typically used for traffic signal timing.
- The 85th percentile speed for free-flowing through vehicles approaching a signalized intersection may be accurately estimated as follows:
  - Add 7 mph to the posted speed limit for approaches with speed limits between 30 and 55 mph.
  - Add 10 mph to the speed limit for approaches with speed limits of 25 mph.

Approach Speeds of Left-Turning Vehicles

- Left-turning vehicles typically approach the intersection at a lower speed than vehicles traveling through the intersection.
- The 85th percentile speed for free-flowing left-turning vehicles approaching a signalized intersection with a posted speed limit between 40 and 55 mph may be accurately estimated by subtracting 5 mph from the posted speed limit.
- Left-turn clearing speeds are limited by the geometry of the turning movement.

RECOMMENDATIONS

The salient findings from this field study provided the basis for several recommendations pertaining to the primary variables for the timing of yellow change and red clearance intervals. The recommendations were specifically formulated for use in NCHRP 03-95 toward development of a comprehensive and uniform set of recommended guidelines for determining the duration of these intervals with the kinematic equation. A detailed description of the resulting recommended guidelines is provided in McGee et al. (*16*).

First, perception–reaction time and deceleration rate should be jointly considered for calculation of yellow change intervals with the kinematic equation. Drivers do not select these variables independently of one another. Slow-reacting drivers tend to compensate with greater deceleration rates, and quick-reacting drivers tend to decelerate more comfortably. In either case, the decision to stop and subsequent braking occur over approximately the same overall time and distance. Parsonson discussed this principle in detail in 1983 (*17*) in a published follow-up discussion to research on driver behavior by Wortman and Matthias (*11*). It follows that the selection of perception–reaction times and deceleration rates should be based on centralized values (e.g., mean or median) for each parameter rather than more extreme values (e.g., 85th percentile or 15th percentile). Thus, the authors recommend a perception–reaction time of 1.0 s and a deceleration rate of 10.0 ft/s^2 for calculating the timing of the yellow change interval with the kinematic equation. These values represent the respective overall means observed in the field study and are consistent with the findings of previous research (*10–13*). They also serve to validate the current default values recommended by ITE (*1, 3*).

The yellow change and red clearance intervals should also be considered jointly when an appropriate approach speed is selected. With the exception of intervals timed for left-turn movements, uniform speed values should be used for determining the duration of both intervals. Specifically, the 85th percentile speed on the approach should be used to determine the timing of yellow change and red clearance intervals. A lower speed should not be used to time the red clearance interval so as to provide additional clearing time for through movements. Drivers approaching at speeds lower than the assumed value will generally have more than adequate time during the yellow interval to either stop comfortably or enter the intersection without accelerating. Providing excessive red clearance time may encourage improper behavior, including red light running. However, a lower speed may be used to time red clearance intervals for left-turn movements.

In lieu of field-measured speed data, the 85th percentile approach speed may be estimated for through movements by adding 7 mph to the posted speed limit on the approach. For locations with speed limits of 25 mph, the option is given to use the speed limit plus

10 mph for estimation of the 85th percentile speed on the approach. The 85th percentile approach speed should be applied to both the yellow change and red clearance intervals for through vehicular movements. For timing of yellow change intervals for left-turn movements, the 85th percentile approach speed may be accurately estimated by subtracting 5 mph from the posted speed limit. A clearing speed of 20 mph may be uniformly applied for timing of red clearance intervals for left-turn movements.

REFERENCES

1. *Traffic Engineering Handbook,* 6th ed. ITE, Washington, D.C., 2009.
2. Gazis, D., R. Herman, and A. Mardudin. The Problem of the Amber Signal Light in Traffic Flow. *Operations Research,* Vol. 8, No. 1, 1960, pp. 112–132.
3. *Determining Vehicle Change Intervals: A Proposed Recommended Practice.* ITE, Washington, D.C., 1985.
4. Eccles, K. A., and M. Liu. *Assessment of Virginia Department of Transportation Policy on Change Intervals.* Virginia Department of Transportation, Richmond, 2008.
5. McGee, H., Sr., K. Moriarty, T. Gates, K. Eccles, R. Retting, and M. Liu. *NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections.* Transportation Research Board of the National Academies, Washington, D.C., in press.
6. Bonneson, J. A., K. H. Zimmerman, and M. Brewer. *Engineering Countermeasures to Reduce Red-Light-Running.* Report 4027-2. Texas Transportation Institute, College Station, 2002.
7. *Uniform Vehicle Code.* National Committee on Uniform Traffic Laws and Ordinances, Strasburg, Va., 2000.
8. Zegeer, C. V., and R. C. Deen. Green-Extension Systems at High-Speed Intersections. *ITE Journal,* Vol. 48, No. 11, 1978, pp. 19–24.
9. Bonneson, J., D. Middleton, K. Zimmerman, H. Charara, and M. Abbas. *Intelligent Detection–Control System for Rural Signalized Intersections.* Report No. 4022-2. Texas Transportation Institute, College Station, 2002.
10. Williams, W. L. Driver Behavior During the Yellow Interval. In *Transportation Research Record 644,* TRB, National Research Council, Washington, D.C., 1977, pp. 75–78.
11. Gates, T. J., D. A. Noyce, L. Laracuente, and E. V. Nordheim. Analysis of Driver Behavior Dilemma Zones at Signalized Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2030,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 29–39.
12. Chang, M.-S., C. J. Messer, and A. J. Santiago. Timing Traffic Signal Change Intervals Based on Driver Behavior. In *Transportation Research Record 1027,* TRB, National Research Council, Washington, D.C., 1985, pp. 20–30.
13. Wortman, R. H., and J. S. Matthias. Evaluation of Driver Behavior at Signalized Intersections. In *Transportation Research Record 904,* TRB, National Research Council, Washington, D.C., 1983, pp. 10–20.
14. *SPSS, Version 18.* The SPSS Inc., Chicago, Ill., 2010.
15. *A Policy on Geometric Design of Highways and Streets,* 5th ed. AASHTO, Washington, D.C., 2004.
16. McGee, H., Sr., K. Moriarty, and T. Gates. Guidelines for Timing Yellow and Red Intervals at Signalized Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2298,* Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 1–8.
17. Parsonson, P. S. Evaluation of Driver Behavior at Signalized Intersections. Discussion. In *Transportation Research Record 904,* TRB, National Research Council, Washington, D.C., 1983, pp. 10–20.

The Traffic Control Devices Committee peer-reviewed this paper.

Closed-Course Evaluation of Stop Paddles with Embedded Lights

Melisa D. Finley, Eun Sug Park, and William Huff

The safety of crossing guards is important because it is their responsibility to make sure that schoolchildren cross the street safely and efficiently. Through discussions, it was found that crossing guards were very interested in using stop paddles with embedded lights to improve the conspicuity of the paddles. There was a concern, however, that some of the embedded light configurations might negatively affect a motorist's ability to recognize the three critical characteristics that define a stop sign: red background color, octagon shape, and white stop legend. A closed-course human factors study was conducted to assess the effectiveness of five stop paddles with embedded lights as compared with that of a standard, unlit stop paddle. On the basis of the findings from this study, the use of three stop paddle configurations is recommended to improve the conspicuity of crossing guards without negatively affecting a motorist's ability to recognize the three critical characteristics of a stop sign. These configurations are (*a*) a stop paddle containing flashing red lights arranged in an octagonal pattern at the eight corners of the paddle, (*b*) a stop paddle containing a series of steady-burn red lights around the border arranged such that the lights clearly convey the octagonal shape of the paddle, and (*c*) a stop paddle containing a series of flashing red lights around the border arranged such that the lights clearly convey the octagonal shape of the paddle. Because the *Manual on Uniform Traffic Control Devices* does not currently address the issue of steady-burn lights, agencies should contact FHWA before they use a stop paddle with a steady-burn light configuration.

The safety of crossing guards is important because they are responsible for the safe and efficient crossing of schoolchildren within crosswalks. In one portion of a recent project sponsored by the Texas Department of Transportation (DOT), researchers from the Texas A&M Transportation Institute (TTI) identified the following three main concerns regarding the safety of crossing guards:

- The most hazardous time for crossing guards is while they are in the active process of helping students to cross the roadway.
- Catching the attention of motorists before a crossing guard steps into the street is critical to improving safety.
- Keeping motorists stopped throughout an active crossing event is imperative.

M. D. Finley and E. S. Park, Texas A&M Transportation Institute, Texas A&M University System, 3135 TAMU, College Station, TX 77843-3135. W. Huff, Wisenbaker Engineering Research Center, Texas A&M University System, 3136 TAMU, College Station, TX 77843-3136. Corresponding author: M. D. Finley, m-finley@tamu.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 22–29.
DOI: 10.3141/2298-03

Given these primary concerns, the decision was made to investigate methods to improve the conspicuity of crossing guards so they are more easily detected within the environment, especially as they enter the crosswalk to stop traffic. It was found that crossing guards were very interested in using stop paddles with embedded lights to improve their conspicuity before entering the crosswalk and during active operations. However, the researchers were concerned that some of the embedded light configurations might negatively affect a motorist's ability to recognize the three critical characteristics that define a stop sign: red background color, octagon shape, and white stop legend. Thus, the decision was made to conduct a closed-course human factors study to determine the effectiveness of five stop paddles with embedded lights as compared with that of a standard, unlit stop paddle.

BACKGROUND

According to the *Manual on Uniform Traffic Control Devices* (MUTCD), stop paddles must be octagonal in shape with a red background and a white border (*1*). The paddle must be at least 18 in. in diameter and have a white stop legend, with at least 6-in. capital letters, on both sides. Stop paddles may be modified to improve conspicuity by incorporating white or red flashing lights on both sides of the paddle. The flashing lights may be arranged in the following configurations:

- Two white or red lights centered vertically above and below the stop legend;
- Two white or red lights centered horizontally on each side of the stop legend;
- One white or red light centered below the stop legend;
- A series of eight or more white or red lights along the outer edge of the paddle, arranged in an octagon pattern at the eight corners of the paddle (more than eight lights may be used only if the arrangement of lights is such that it clearly conveys the octagon shape of the paddle); and
- A series of white lights forming the shapes of the letters in the legend.

When flashing lights are used, the flash rate shall be at least 50 flash periods per minute (fpm) but not more than 60 fpm.

Although earlier research concerning the use of embedded lights to enhance normal stop signs and stop–slow paddles operated by flaggers in work zones was found, there was no documentation of previous research concerning the use of embedded lights to enhance stop paddles operated by crossing guards in school zones.

Stop Signs

In a previous project performed by TTI, Gates et al. investigated the effect of various higher-conspicuity sign materials on traffic operations and motorist behavior, including a normal stop sign with eight flashing red LEDs embedded in the corners of the sign (*2*). They collected and analyzed traffic operations data at field sites before and after the installation of the LED stop signs. The research found that the flashing red LED stop signs were beneficial in reducing stop sign violations (i.e., blow-throughs and roll-throughs) during both daytime and nighttime conditions. However, the flashing red LED signs did not affect vehicle speeds or decelerations on the approach to the intersections.

Arnold and Lantz also investigated the use of normal stop signs with eight flashing red LEDs embedded at each corner (*3*). As in the TTI study, they collected data on vehicles approaching the stop sign both before and after installation of the LED signs. They concluded that the flashing red LED stop signs were effective in reducing average speeds of vehicles approaching an intersection. However, the effects of LED signs on compliance were inconclusive.

Stop–Slow Paddles

Stop–slow paddles are similar to stop paddles, except one side has a black border around an orange background with a black legend that reads "slow." In 1996, Agent and Hibbs evaluated the use of various work zone devices developed by the SHRP, one of which was a flashing handheld stop–slow paddle (*4*). They evaluated six different models of stop–slow paddles; however, all of the paddles contained one or two flashing white lights (either halogen or strobe) on the stop face of the sign. Agent and Hibbs found the overall experience with the flashing stop–slow paddles to be very positive; motorists were able to observe the flashing paddle sooner than a standard unlit paddle. Workers had varied opinions about the different flashing paddles, mainly concerning cost, durability, reliability, and usability of the paddles.

Morena also evaluated five different flashing stop–slow paddles for a group of federal, state, and local highway safety workers (*5*). Observers thought that, in bright sunlight, the paddle with halogen lights was the only paddle that would be able to draw a motorist's attention from 285 ft away or farther. Subsequent tests with two halogen paddles confirmed these findings.

More recently, Schrock et al. evaluated the usefulness and workability of several technology-enhanced flagging devices, including three handheld LED-enhanced stop–slow paddles and a standard, unlit paddle (*6*). All of the stop–slow paddles were 24 in. in diameter and fabricated with engineer-grade sign sheeting (ASTM Type I). The following LED lighting configurations on the stop face of the sign were tested:

- Flashing red lights at the eight corners of the paddle,
- A series of flashing red lights forming the shapes of the letters in the stop legend (not currently included in the MUTCD), and
- Flashing red lights centered vertically above and below the stop legend.

Schrock et al. used responses from maintenance personnel in focus groups, as well as field surveys of motorists who viewed the devices in active work zones, to gauge the usefulness of each device. Although maintenance personnel thought that the flashing lights would increase paddle conspicuity, they expressed concern regarding the durability and usability of the LED-enhanced paddles, citing fragile components, increased weight, and the need to recharge a paddle's batteries. Only 15% of surveyed motorists stated that they saw the stop–slow paddle (independent of the type); the distribution among the four types of paddles was similar. Each motorist only saw one type of paddle. Of those surveyed, 54% did not think the flashing lights indicated a more important situation, and only 26% of motorists thought they drove differently when they saw the LED-enhanced stop–slow paddles (e.g., drove slower, slowed down earlier, became more cautious). Motorists liked the stop–slow paddle with the red flashing LED lights above and below the stop legend the best (79%), closely followed by the stop–slow paddle with red lights forming the stop legend (72%). Motorists disliked the stop–slow paddle with the red lights at the eight corners the most (38%), because they did not see it or found it difficult to see.

EXPERIMENTAL DESIGN

A closed-course human factors study was conducted on the roadway network at the Texas A&M University Riverside Campus, which is a 2,000-acre complex of research and training facilities located 10 mi northwest of the university's main campus. The decision to conduct the study on the roadway network was made because the roadway network more closely replicated an urban environment with driveways and cross streets but still provided less traffic than an actual urban street. Although data collected in response to simulated crossing-guard operations cannot be directly compared with data collected during an actual crossing-guard operation at a school, closed-course study data can be used to compare the relative differences in performance among the various treatments evaluated. The following sections describe the treatments, time periods, study procedure, and participants.

Treatments

An extensive Internet search and discussions with manufacturers revealed that not all of the light configurations listed in the MUTCD were readily available. The researchers could not identify a current manufacturer of a stop paddle with lights centered horizontally on each side of the stop legend or of a stop paddle with one light centered below the stop legend, even though the latter device appeared to have been developed during the SHRP study (*4*). It also was not possible to find any manufacturers that had off-the-shelf stop paddles with white lights, except in the letters of the stop legend. Several manufacturers that offer steady-burn lights within stop paddles were discovered, even though currently the MUTCD addresses the addition of only flashing lights to improve conspicuity. On the basis of time, budget, and off-the-shelf availability constraints, the decision was made to include the following devices in the study:

- Standard, unlit stop paddle;
- Stop paddle containing eight flashing red LED lights around the border, one in each corner;
- Stop paddle containing a series of steady-burn red LED lights around the border;
- Stop paddle containing a series of flashing red LED lights around the border;

- Stop paddle containing flashing red LED lights centered vertically above and below the stop legend; and
- Stop paddle with a series of steady-burn white LED lights forming the letters of the stop legend.

All paddles were 18 in. in diameter (the minimum size required by the MUTCD) and fabricated with high-intensity sign sheeting (ASTM Type III). Figure 1 shows the six stop paddle designs. Treatment 2 and Treatment 3b both produced approximately 60 fpm. For Treatment 4, the top lights flash twice and then the bottom lights flash twice. A flash period was assumed to be from the beginning of the top lights' flash sequence to the end of the bottom lights' flash sequence, which yielded approximately 70 fpm.

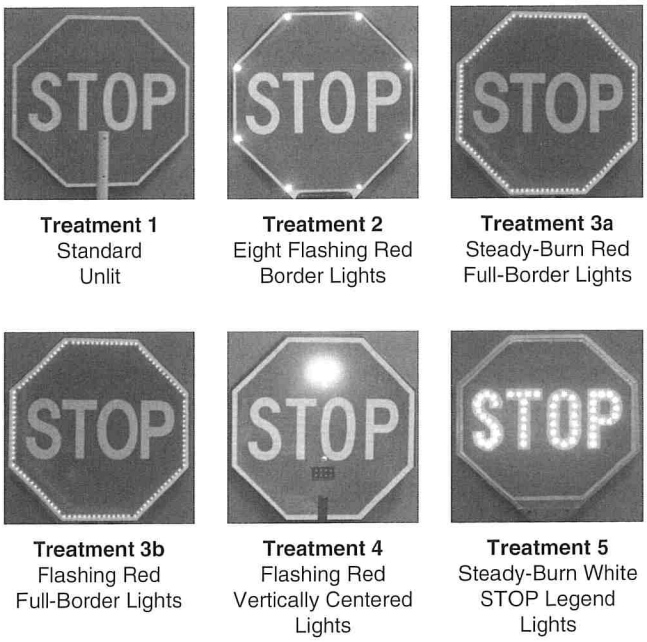
Because the primary concern was the safety of crossing guards when the guards begin to enter the travel lanes from the shoulder to stop traffic, the researcher holding each treatment was positioned on the left or right side of the road instead of in the middle of the roadway. This researcher wore a fluorescent yellow-green vest and displayed only one treatment at a time.

To reduce learning effects, six different treatment orders were used. Each participant saw each treatment once on either the left or the right side of the road. Thus, approximately half of the participants saw the treatments on the left side of the road and the other half saw the treatments on the right side of the road. The road chosen was ideal for the study because it had fair pavement conditions, no stop signs at the cross streets, a low volume of traffic in the surrounding area, and a favorable direction in relation to the sun.

Time Periods

In discussions with crossing guards, it was found that the guards who work of during early morning hours and those located directly in front of the sun in the morning (i.e., those who were backlit) experienced more motorist noncompliance. This noncompliance was believed to occur because there are low-visibility conditions before sunrise and the sun "blinds" motorists when it is near the horizon early in the morning. Therefore, the study was conducted during low-light conditions and when the sun was near the horizon. To complete the study in a timely manner, the researchers collected data during both morning and evening periods by reversing the travel direction along the selected roadway.

According to previously conducted field observations, the earliest school zone time started at 7:00 a.m. This period was compared with the sunrise times for the entire year to determine the proportion of time that crossing guards were active during twilight and early morning periods (*7*). On the basis of this information and the sunrise–sunset times during the study period, the researchers decided to conduct the morning portion of the study from 6:00 to 7:00 a.m. (i.e., 30 min before and after sunrise) and the evening portion of the study from 8:00 to 9:00 p.m. (i.e., 30 min before and after sunset). The actual study times were 6:00 to 6:45 a.m. (sunrise approximately 6:30 a.m.) and 8:00 to 8:45 p.m. (sunset approximately 8:30 p.m.). Every attempt was made to collect data during the same periods each day. During each period, participants drove toward the sun (i.e., east in the morning and west in the evening). However, for half of the data collection periods, there were overcast weather conditions.



Treatment 1 Standard Unlit

Treatment 2 Eight Flashing Red Border Lights

Treatment 3a Steady-Burn Red Full-Border Lights

Treatment 3b Flashing Red Full-Border Lights

Treatment 4 Flashing Red Vertically Centered Lights

Treatment 5 Steady-Burn White STOP Legend Lights

FIGURE 1 Stop paddle treatments.

Study Procedure

Upon arrival, each participant checked in and a briefing took place at a TTI office building. Participants then completed screenings for standard visual acuity, contrast sensitivity, and color blindness. These screenings provided comparison information for data reduction and ensured that all participants had at least minimal levels of acceptable vision before they began the study. No participants were disqualified from the study on the basis of these screenings.

Participants were told that they would drive an instrumented state-owned passenger vehicle (i.e., Ford Taurus) on the roadway network where researchers had set up a simulated school crosswalk. They were told to obey all traffic control devices and to drive at the posted speed limit (i.e., 30 miles per hour).

In the planning of the study, it was determined that all of the LED-enhanced stop paddles could be detected further away than the standard, unlit stop paddle. Thus, the detection distance of the stop paddle treatments (i.e., a measure of conspicuity) was not evaluated. As previously indicated, the researchers were interested in assessing the impact of the embedded lights on a motorist's ability to recognize the red background color, octagon shape, and white stop legend of the stop paddles. Because of the design of the LED-enhanced stop paddles, the legend could not be altered (e.g., to read "tops") to evaluate legibility. In addition, it was acknowledged that the majority of motorists already know the background color, shape, and legend of a stop sign. Therefore, participants were informed that they would need to identify these three characteristics of a stop paddle before the study. Participants were instructed to verbally acknowledge when they could clearly identify the red background color of the sign and the octagon shape of the sign and completely read the stop legend on the sign. Participants were also instructed not to guess or assume any of the responses but to make sure they could clearly see the desired characteristic before verbally acknowledging it.

The following data were recorded: background color recognition distance, shape recognition distance, legend legibility distance, and any comments made by the participants. The participants were

instructed not to stop but to continue traveling straight past the simulated crossing guard. After each treatment, participants were asked a series of follow-up questions. Each participant repeated this process for each stop paddle treatment. At the end of the study, participants were asked to rank the six treatments from best to worst with regard to their ability to catch motorists' attention and inform them to stop.

Participants

Thirty-six people from the area around Bryan–College Station, Texas, were recruited to participate in the closed-course study. Participants had to be at least 18 years old and have a current Texas driver's license. Half of the participants were male and half were female. In each gender group, 31% of the participants were 18 to 39 years old and 19% were at least 55 years old. The demographic sample obtained was based on the gender and age of the Texas driving population (*8*).

DATA REDUCTION AND ANALYSIS

Following the data collection, each participant's raw data were screened and reduced into a fully formatted data set to obtain the necessary information for analysis. During the data-screening process, any anomalous data (e.g., misidentifications, malfunctioning treatment) were eliminated.

Table 1 contains the variables considered in the analyses. Three response variables were considered in order to assess the effectiveness of the stop paddles; the effect of other variables of interest (i.e., treatment, time period, roadside location, age, and gender) was also considered. A split-plot design analysis was used to cope with driver-to-driver variability, with participant as a whole plot and each treatment combination as a split plot. The demographic variables (gender and age) served as whole-plot factors; the treatment, time period, and roadside location served as split-plot factors. Separate analyses were conducted for each of three response variables. A 5% significance level ($\alpha = 0.05$) was used for all statistical analyses.

TABLE 1 Variables Used in the Analysis

Class	Variable	Data Set Values
Response variables	Background color recognition distance (ft)	20~1,770
	Shape recognition distance (ft)	128~1,313
	Legend legibility distance (ft)	118~908
Factors	Treatment	Standard, unlit
		Eight flashing red border lights
		Steady-burn red full-border lights
		Flashing red full-border lights
		Flashing red vertically centered lights
		Steady-burn white "Stop" legend
	Time period	Morning
		Evening
	Roadside location	Left
		Right
	Age category (years)	18 to 39
		≥55
	Gender	Male
		Female
	Participant	1, 2, . . . , 36

The participants' subjective opinions regarding the advantages and disadvantages of each treatment were also analyzed. The researchers calculated the average rating for each treatment with respect to how well the participant could recognize and read the paddle. With a score of 1 being very well, 2 being well, and 3 being not well, a treatment perceived to do well would have a lower average rating. In addition, the overall ranking of the treatments from best to worst was computed with a rank score: 1 point was given each time a treatment was ranked first (best), 2 points were given for second-place rankings, and so on, with the maximum number of points assigned each time a treatment was ranked worst. Thus, the treatment perceived to be the best would have the lowest score.

RESULTS

The following subsections contain the results of the analyses on the background color recognition distance, shape recognition distance, and legend legibility distance. The predicted values (least squares means) for each response variable were used to compare different treatments. When there are multiple factors in the model, comparisons should not be made between raw cell means in data, because raw cell means do not compensate for other factors in the model. The least squares means are the predicted values of the response variable for each level of a factor that have been adjusted for the other factors in the model. Trends found in the subjective data are also reported.

Background Color Recognition

A model was initially fit with all main effects and possible two-way interactions. The restricted maximum likelihood (REML) method was used to obtain the results of the statistical analysis, which showed that the two-way interaction between age and time period was statistically significant ($\alpha = 0.05$). Tukey's honestly significant difference (HSD) multiple comparison procedure showed that, during the evening period, the predicted distance for background color recognition for the younger age category (823 ft) was statistically farther than that for the older age category (616 ft). During the morning period, however, the difference between the two age categories was not statistically significant. Based on these data, the final model included all of the main effects (i.e., treatment, time period, roadside location, age category, and gender) and the two-way interaction between age and time period as fixed effects, and participants nested within age category and gender as random effects. The final model found the treatment to be the only statistically significant factor ($\alpha = 0.05$) other than the aforementioned two-way interaction between age and time period.

Figure 2 presents the predicted means for the distance for background color recognition for each treatment. Tukey's HSD procedure was used to determine which treatment factor levels were statistically different. Treatment 2 was found to have the farthest predicted mean distance for background color recognition (882 ft). However, this distance was not significantly different from the predicted mean distances for background color recognition for Treatments 1, 3a, and 3b (802 ft, 853 ft, and 802 ft, respectively).

The predicted mean distances for background color recognition for Treatments 4 and 5 (592 ft and 490 ft, respectively) were

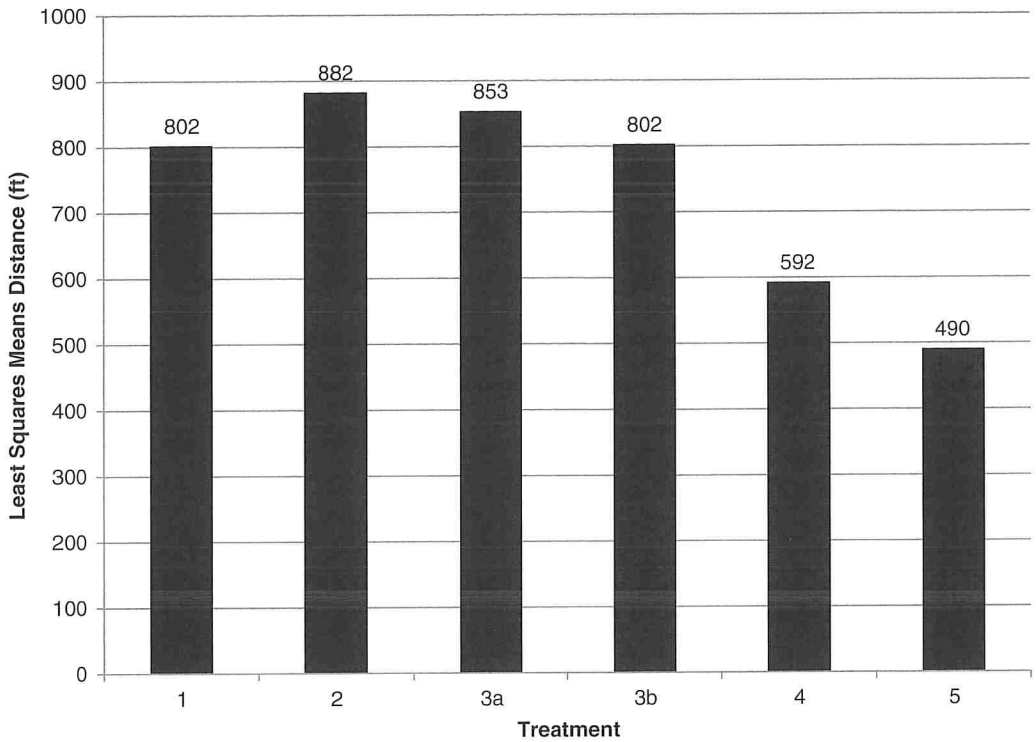


FIGURE 2 Comparison of data on distance of background color recognition, by treatment. Treatments 4 and 5 were significantly different from all other treatments.

found to be significantly shorter than those for Treatments 1, 2, 3a, and 3b. According to the predicted means, participants traveled 210 to 392 ft closer to the crossing guard before being able to recognize the red background color of the stop paddle for Treatments 4 and 5. Thus, the light configurations in Treatments 4 and 5 did appear to negatively affect a motorist's ability to recognize the red background color of the stop paddle compared with the standard, unlit stop paddle.

The subjective data for Treatment 4 showed that 17% of participants thought red flashing lights above and below the stop legend were blinding or distracting. In addition, 6% of the participants specifically stated that these lights made it difficult to discern the red background color of the stop paddle. The subjective data for Treatment 5 showed that 42% of participants thought that the white lights forming the stop legend made it more difficult to distinguish the red background color of the stop paddle.

Shape Recognition

A model was initially fit with all main effects and possible two-way interactions. The statistical analysis results obtained by the REML method showed that none of the two-way interaction effects were statistically significant. Therefore, the final model included all of the main effects (i.e., treatment, time period, roadside location, age category, and gender) as fixed effects and participants nested within age category and gender as random effects. The final model found the treatment to be the only statistically significant factor ($\alpha = 0.05$).

Figure 3 presents the predicted means for the shape recognition distance for each treatment. Tukey's HSD procedure was used to determine which treatment factor levels were statistically different. Treatment 3b had the farthest predicted mean distance for background shape recognition (559 ft). However, this distance was not significantly different from the other treatments, with the exception of Treatment 5 (364 ft). Treatments 2 and 3a were also found to have a predicted mean distance for shape recognition significantly greater than that of Treatment 5. The subjective data for Treatment 5 showed that 25% of participants thought the white lights forming the stop legend made it more difficult to identify the octagon shape of the stop paddle. The predicted mean distances for shape recognition for Treatments 1 and 4 (449 ft and 455 ft, respectively) were not significantly different from any of the other stop paddles studied. Although there were significant differences among some of the stop paddles with embedded lights, none of the stop paddles with embedded lights appear to negatively affect a motorist's ability to recognize the octagon shape of the stop paddle compared with the standard, unlit stop paddle.

Legend Legibility

As with the other two response variables, a model was initially fit with all main effects and possible two-way interactions. The statistical analysis results obtained by the REML method showed that the age and time period and treatment and time period two-way interactions were statistically significant ($\alpha = 0.05$). However, the effect of the age and time period interaction did not seem to be practically significant, and Tukey's HSD procedure did not find any of the levels of the two-way interaction to be significantly different. Therefore, the age and time period interaction was not retained in the final model. The final model included all the main effects (i.e., treatment, time period, roadside location, age category, and

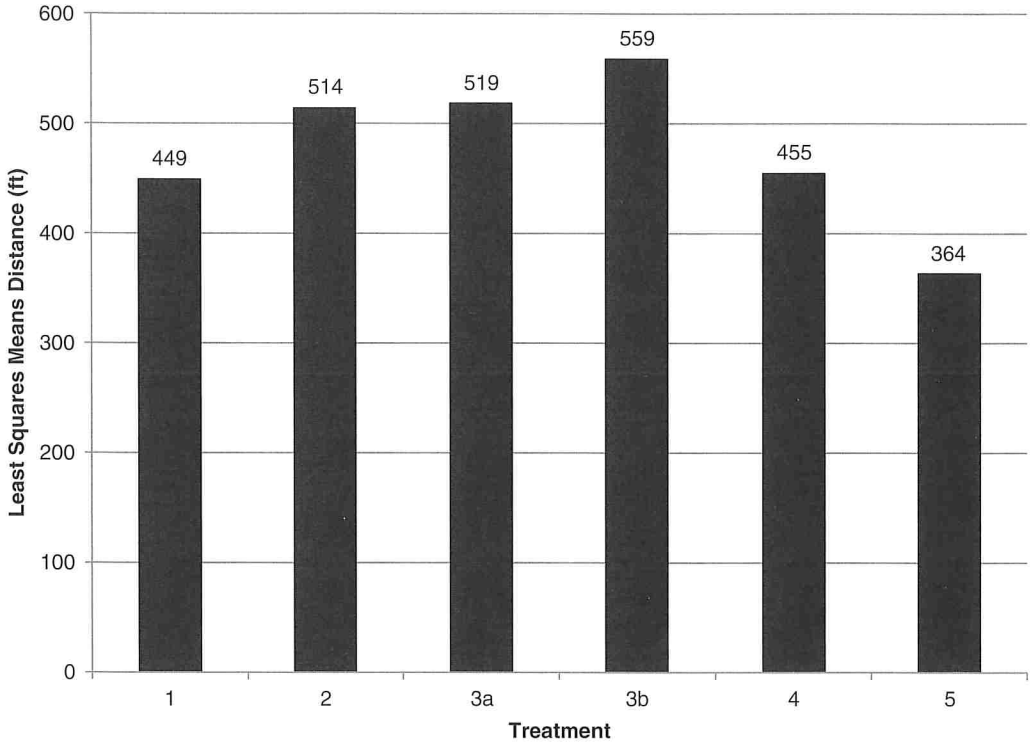


FIGURE 3 Comparison of data on distance of shape recognition, by treatment. Treatment 5 was significantly different from Treatments 2, 3a, and 3b.

gender) and the two-way interaction between treatment and time period as fixed effects, and participants nested within age category and gender as random effects. The final model found the treatment, as well as the treatment and time period interaction, to be statistically significant ($\alpha = 0.05$).

Because the interaction effect between treatment and time period was significant, the effect of treatment was assessed for each time period (i.e., morning and evening). Figure 4 presents the predicted means for the legend legibility distance for each treatment during each time period. Tukey's HSD procedure was used to determine which treatment and time period factor level combinations were statistically different. For the morning period, there were no statistically significant differences among treatments. For the evening period the predicted mean legibility distance for Treatment 5 (463 ft) was significantly longer than that for Treatments 3a and 4 (351 ft and 362 ft, respectively), but it was not statistically different from the other treatments. For Treatment 4, 50% of the participants thought that the flashing red lights vertically centered above and below the stop legend made it difficult to read the legend. In contrast, for Treatment 3a only 11% of the participants thought that the steady-burn red border lights negatively affected their ability to read the stop legend.

Also, as shown in Figure 4, the time period affected only the predicted mean legibility distance of Treatment 5. In the evening period, the predicted legibility distance was 116 ft farther than in the morning period. During two-thirds of the evening data collection periods the weather was clear and sunny. Under these conditions, the light produced by Treatment 5 was most likely weaker than the ambient light conditions and the treatment would have been backlit by the setting sun, thus decreasing the potential for overglow. In contrast, during two-thirds of the morning data collection periods the weather was overcast. Under predawn and overcast conditions, the light produced by Treatment 5 was most likely stronger than the ambient light conditions that resulted in overglow of the stop legend. The subjective data support this hypothesis: 31% of participants stated that Treatment 5 was more difficult to read because the white lights were too bright and blended together (i.e., overglow). Although the time period appears to have affected the predicated mean legibility distance of Treatment 5, Tukey's HSD procedure found that the difference was not statistically significant.

ADDITIONAL SUBJECTIVE DATA

As discussed previously, the participants rated each treatment independently according to their ability to recognize and read each stop paddle. The participants also ranked the six treatments from best to worst on their ability to catch a motorist's attention and inform them to stop. Table 2 contains the mean rating and rank score for each treatment. Treatments 2, 3a, and 3b (stop paddles with lights around the border) were rated and ranked the best. The primary reason provided by participants was that these stop paddles catch a motorist's attention but do not distract from the background color, shape, and legend.

SUMMARY AND RECOMMENDATIONS

A closed-course human factors study was conducted to determine the effectiveness of five stop paddles with embedded lights compared with a standard, unlit stop paddle. Before the study, the researchers verified that the addition of lights improved the conspicuity of the

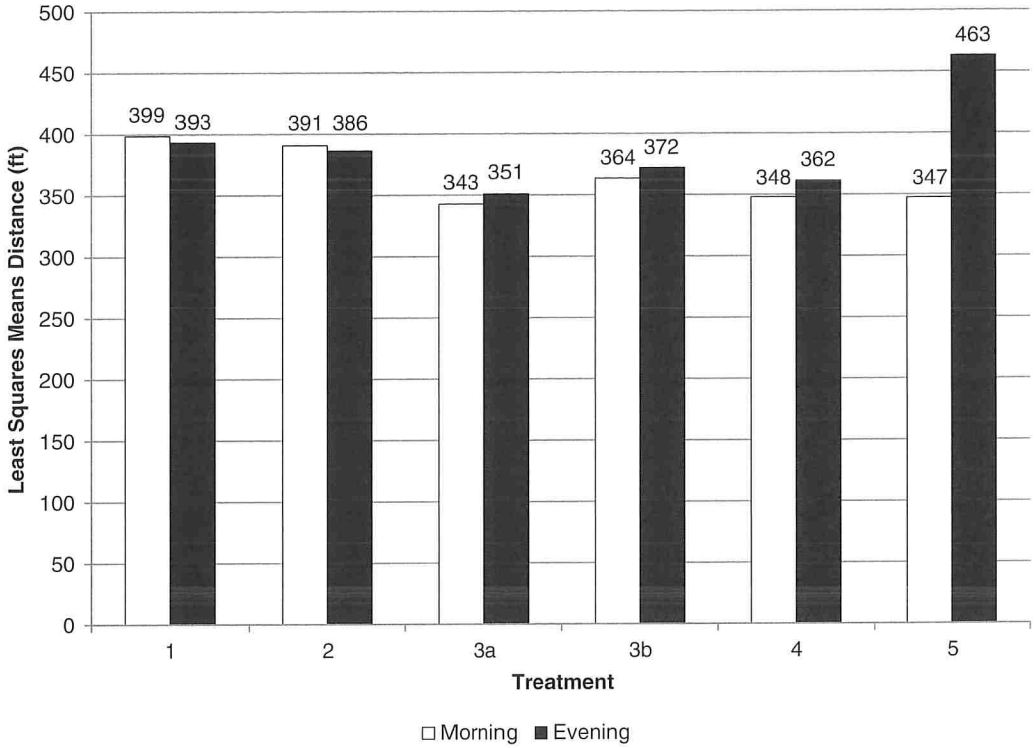


FIGURE 4 Comparison of data on distance of legend legibility, by treatment and time period. Treatments 3a and 4 were significantly different from Treatment 5 during the evening period.

crossing guard (i.e., a lit paddle could be detected farther away than the standard, unlit stop paddle). Thus, the study evaluated the impact of lights on a motorist's ability to recognize the three critical characteristics that define a stop sign: red background color, octagon shape, and white stop legend.

The three stop paddles with lights around the border did not significantly affect the predicted mean distance at which the participants could recognize the background color of the stop paddles compared with the standard, unlit stop paddle. In contrast, the stop paddle with flashing red lights vertically centered above and below the stop legend and the stop paddle with steady-burn white lights forming the stop legend resulted in predicted mean distanced for background color recognition that were significantly shorter than those for the standard, unlit stop paddle (210 to 392 ft closer to the crossing guard). Participants thought the flashing lights above the stop legend were blinding or distracting and thus made it difficult to discern the red background color of the stop paddle. Participants also thought the white lights forming the stop legend made it more difficult to distinguish the red background color of the stop paddle.

TABLE 2 Subjective Opinion Data

Treatment	Mean Rating[a]	Rank Score[b]
1	1.50	170
2	1.42	115
3a	1.33	104
3b	1.47	78
4	1.64	149
5	2.08	140

[a]The lowest rating, 1, corresponded with the highest preference.
[b]Treatment perceived to be best would have lowest score.

None of the stop paddles with embedded lights affected the predicted mean distance at which the participants could recognize the shape of the stop paddles compared with the standard, unlit stop paddle. However, the three stop paddles with lights around the border did produce significantly farther predicted mean distances for shape recognition (150 to 195 ft farther from the crossing guard) than the stop paddle with steady-burn white lights forming the stop legend. This is not surprising because the lights around the border highlight the shape of the stop sign.

None of the stop paddles with embedded lights affected the predicted mean distance at which the participants could read the stop legend compared with the standard, unlit stop paddle. Although the finding was not statistically significant, researchers did find that ambient lighting conditions affected the legibility distance of the stop paddle with steady-burn white lights forming the stop legend. Under clear and sunny conditions, the predicted mean legibility distance for this stop paddle was 463 ft. In contrast, under predawn and overcast conditions, the predicted mean legibility distance was 116 ft shorter (i.e., 347 ft). The researchers believe that when the light produced by the steady-burn white lights in the stop legend is greater than the ambient light, overglow of the stop legend occurs (i.e., blending of the letters), making the legend more difficult for motorists to read.

The use of the following stop paddles is recommended to improve the conspicuity of crossing guards without negatively affecting a motorist's ability to recognize the three critical characteristics of

a stop sign (i.e., red background color, octagon shape, and white stop legend):

- A stop paddle containing flashing red lights arranged in an octagonal pattern at the eight corners of the paddle,
- A stop paddle containing a series of steady-burn red lights around the border arranged such that the lights clearly convey the octagonal shape of the paddle, and
- A stop paddle containing a series of flashing red lights around the border arranged such that the lights clearly convey the octagonal shape of the paddle.

Because the MUTCD does not currently address steady-burn lights, agencies should contact FHWA before they use a stop paddle with a steady-burn light configuration. The researchers do not know how the use of white lights would affect the effectiveness of the recommended stop paddles.

On the basis of the findings, use of the following devices is not recommended:

- A stop paddle containing flashing red lights centered vertically above and below the stop legend and
- A stop paddle with a series of steady-burn white LED lights forming the letters of the stop legend.

The study did not evaluate the latter treatment with flashing lights, as described in the MUTCD. Thus, the researchers do not know whether the use of flashing lights would reduce the potential for overglow and thus improve a motorist's ability to recognize the background color and shape, as well as read the stop legend, during darker ambient light conditions.

The study did not evaluate the effectiveness of a stop paddle containing flashing red lights centered horizontally on each side of the stop legend. However, the researchers believe that stop paddles containing a light configuration similar to the one tested above and below the stop legend would also reduce the recognition distance of the background color of the stop paddle. The researchers do not know how the use of white lights would affect the effectiveness of the stop paddles with flashing lights centered horizontally on each side or vertically above and below the stop legend.

ACKNOWLEDGMENTS

The authors thank the Texas DOT and FHWA for sponsoring the research summarized in this paper. The authors also thank the following Texas DOT employees who provided guidance and expertise in various phases of the project: Michael Chacon, Wade Blackmon, Joni Hutson, Timothy Lowak, Grady Mapes, Carlos Rodriguez, David Selman, Gary Tarter, and Wade Odell. In addition, the authors acknowledge the many individuals at TTI who provided valuable assistance during the project.

REFERENCES

1. *Manual on Uniform Traffic Control Devices.* FHWA, U.S. Department of Transportation, 2009. http://mutcd.fhwa.dot.gov/. Accessed July 20, 2011.
2. Gates, T., H. G. Hawkins, Jr., S. T. Chrysler, P. J. Carlson, A. J. Holick, and C. H. Spiegelman. *Traffic Operational Impacts of Higher-Conspicuity Sign Materials.* Report 4271-1. Texas Transportation Institute, College Station, Oct. 2003.
3. Arnold, E., and K. E. Lantz, Jr. *Evaluation of Best Practices in Traffic Operations and Safety: Phase I: Flashing LED Stop Sign and Optical Speed Bars.* Final Report VTRC 07-R34. Virginia Transportation Research Council, Charlottesville, June 2007.
4. Agent, K., and J. O. Hibbs. *Evaluation of SHRP Work Zone Safety Devices.* Report KTC-96-30. Kentucky Transportation Center, Lexington, Dec. 1996.
5. Morena, D. Halogen Stop/Slow Paddle Outshines the Rest. *Crossroads,* Winter 2002, p. 2.
6. Schrock, S., C. F. See, J. Saadi, W. K. Chong, and Y. Bai. *Evaluation of Technology-Enhanced Flagger Devices: Focus Group and Survey Studies in Kansas.* Report 48360-1. University of Kansas, Lawrence, April 2009.
7. U.S. Naval Observatory. *Rise and Set for the Sun for 2010.* Astronomical Applications Department. http://www.usno.navy.mil/USNO/astronomical-applications/data-services. Accessed Aug. 31, 2011.
8. Highway Statistics 2008: Licensed Male Drivers, by Age 1/2008. Table DL-22. FHWA, U.S. Department of Transportation, 2010. http://www.fhwa.dot.gov/policyinformation/statistics/2008/dl22.cfm. Accessed Aug. 31, 2011.

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official view or policies of FHWA or the Texas DOT.

The Traffic Control Devices Committee peer-reviewed this paper.

Detection, Control, and Warning System for Mitigating Dilemma Zone Problem

Karl Zimmerman, Devendra Tolani, Roger Xu, Tao Qian, and Peter Huang

High-speed signalized intersections present a grave hazard to the traveling public. An important component of this hazard is variation in the process of deciding whether to stop or proceed that occurs in the dilemma zone. To mitigate the dilemma zone problem, an innovative vehicle-specific detection, control, and warning system (DCWS) is being developed. To reduce dilemma zone conflicts, DCWS uses detection of individual vehicle speeds to provide unique, speed-specific dilemma zone protection for each vehicle as it approaches a signalized intersection. Although protecting all vehicles would be ideal, under some situations such protection is not possible. To account for these situations, DCWS also uses a vehicle-specific in-pavement system to give advance warning to drivers who are going to be trapped in the dilemma zone. With hardware-in-the-loop simulations, the new DCWS outperforms conventional dilemma zone warning systems in terms of the number of vehicles in the dilemma zone, the number and percentage of vehicles requiring warning, and the number of warning events per hour.

At high-speed signalized intersections, one of the major problems for drivers who are presented with a yellow indication is the decision to stop or proceed. Near the intersection there is an area on each high-speed approach where drivers have the most difficulty deciding (as a group) to proceed or stop, which increases the risks of intersection-related collisions. This dangerous area has been noted as far back as 1960 (*1*).

As individuals approaching an intersection, drivers tend to make a quick decision about whether to proceed or stop at an acceptable deceleration rate (less than approximately 0.3 *g* or 10 ft/s^2) (Figure 1*a*). However, each driver sees the situation differently; as a group, drivers may make a wide range of decisions about whether to proceed or stop. This collective uncertainty is reflected in distributions such as the one shown only for illustrative purposes in Figure 1*b*. The collective uncertainty also creates higher collision risks for these drivers compared with others, especially rear-end and right-angle collisions.

The area of highest uncertainty shown in Figure 1 has been termed the "dilemma zone." The most common definition of the dilemma zone is the one coined by P. S. Parsonson and Technical Committee 18 of ITE in 1974: the area on the approach between where 90% of drivers stop and where 10% of drivers stop when presented with a yellow indication (*2*). This area has also been termed the "decision zone" or "indecision zone" by others, although those terms have no achieved general acceptance. Research has found that, for passenger vehicles, the dilemma zone is approximately 3 s wide, begins 5 to 6 s from the intersection, and ends 2 to 3 s from the intersection (*3–6*). More recent research has found that the dilemma zone for trucks begins and ends about 1.5 s farther from the intersection, as shown in Figure 2 (*7–9*). With the large difference in mass between passenger vehicles and trucks, and because truck drivers often decide to proceed when farther from the intersection than car drivers do, the likelihood of car–truck collisions increases. These collisions are likely to have severe consequences for the passenger car's occupants.

K. Zimmerman, College of Engineering, Valparaiso University, Valparaiso, IN 46383. D. Tolani, R. Xu, and T. Qian, Intelligent Automation Inc., 15400 Calhoun Drive, Suite 400, Rockville, MD 20855. P. Huang, Turner–Fairbank Highway Research Center, Federal Highway Administration, 6300 Georgetown Pike, McLean, VA 22101. Corresponding author: K. Zimmerman, Karl.Zimmerman@valpo.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 30–37.
DOI: 10.3141/2298-04

TRADITIONAL APPROACHES TO DILEMMA ZONE PROTECTION

Two approaches have traditionally been used for dilemma zone protection. The first is to keep the dilemma zone clear if at all possible. To do this, detection is placed upstream of the intersection within the dilemma zone itself for a particular speed range. Two examples, used by different states for this purpose, are shown in Figure 3 (*6, 10*). Other similar types of designs are also used. In theory, if the dilemma zone is completely protected, then drivers will never have to make a decision without clear guidance. In practice, the situation is more complicated: first, the dilemma zone is based on time, not distance; second, there is the issue of max-out.

The primary feature of the dilemma zone is that it is based on travel time, not fixed distances. The upstream and downstream ends of the dilemma zone will vary depending on the approach speed. However, dilemma zone detection is typically based on inductive loops, which are placed at a fixed distance. The detection cannot be varied to account for individual vehicle speed, so the protection may be either too little or too much for any individual vehicle. Too little protection is potentially unsafe; too much protection creates more delay for other movements, wasting time and increasing fuel consumption and emissions. To avoid these problems, a different approach was taken by Bonneson et al. with their detection-control system (D-CS), also shown in Figure 3, which has detection to measure each vehicle's speed and then tailor dilemma zone protection specifically for that vehicle (*11*). The intended result is protection that is just right for all vehicles approaching the intersection, with less wasted time.

All of the designs shown in Figure 3 are vulnerable to max-out, even at relatively moderate traffic volumes, depending on detector

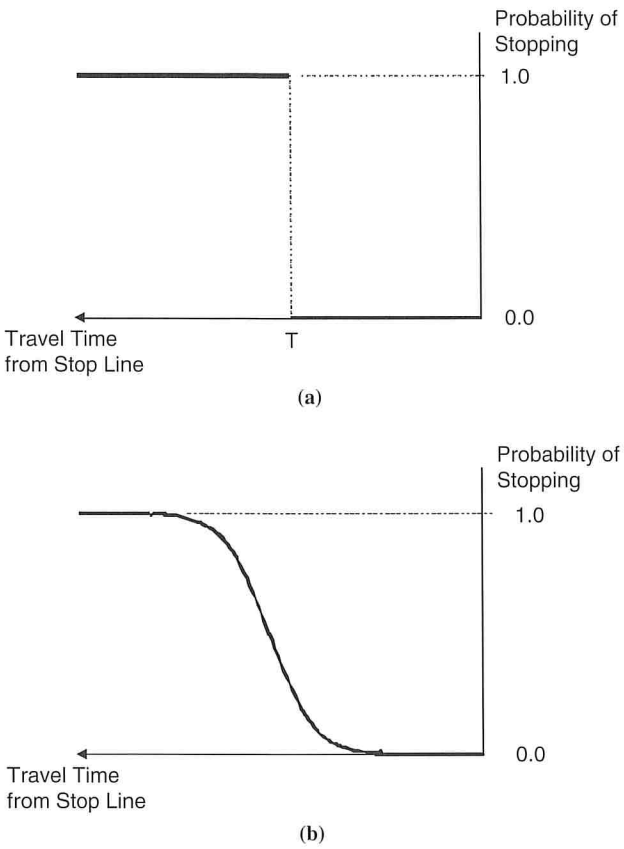


FIGURE 1 Driver decisions to proceed or stop at end of the green phase: (*a*) individual driver's decision and (*b*) aggregate drivers' decisions.

and phase timings. A signal indication cannot be allowed to run forever. Traffic on conflicting movements must be served as well. Controllers place a time limit on the length of time that a green indication may remain on when there is a conflicting call for service, and max-out occurs when that time limit is reached. At max-out, dilemma zone protection ceases, creating the very situation that the dilemma zone protection intended to avoid. In addition, because of the nature of actuated signal operations, at least one vehicle is virtually guaranteed to be in the dilemma zone at max-out, so avoiding max-out is important for intersection safety. The two state systems shown in Figure 3 do not have any max-out protection. D-CS attempts to reduce max-out frequency by intelligently ending the green indication before max-out by choosing a time to end the green indication when there are as few vehicles in the dilemma zone in all lanes as possible, based on a limited look-ahead time into the future. However, even D-CS cannot avoid max-out under some circumstances: perfect dilemma zone protection is not possible.

The third approach to dilemma zone protection has been to warn drivers that the green indication is about to end. This warning is typically done with a standard W3-4 sign with flashing lights. The flashing lights may be active all of the time or active only near the end of the green indication, depending on state policies (*12–15*). The warning does not tell drivers to stop. Instead, the warning provides more information to aid the driver's decision-making process. The devices that are active near the end of the green indication appear to be more effective. However, an actuated traffic signal controller does not know exactly when it will end a green indication until it is called to end. A more effective warning system would be able to look into the future and provide warnings dynamically before the end of the green indication. Sunkari et al. (*16*) developed the advance warning of end-of-green system (AWEGS) to make this type of prediction and warning.

With the maturity and low cost of many new information technologies in the 21st century, especially in the fields of wireless communications, microprocessing boards, and LEDs, there are opportunities to develop a new system that uses these new technologies to further improve the dilemma zone protection techniques and lower the costs for deploying dilemma zone protection systems. FHWA launched a Small Business Initiative Research project in 2010 that uses the new technologies to develop a new dilemma zone protection system, that would integrate and improve the existing techniques with the addition of new concepts of operation and new technologies, including wireless communications, field microprocessing boards, LED flashers, and some new software and firmware.

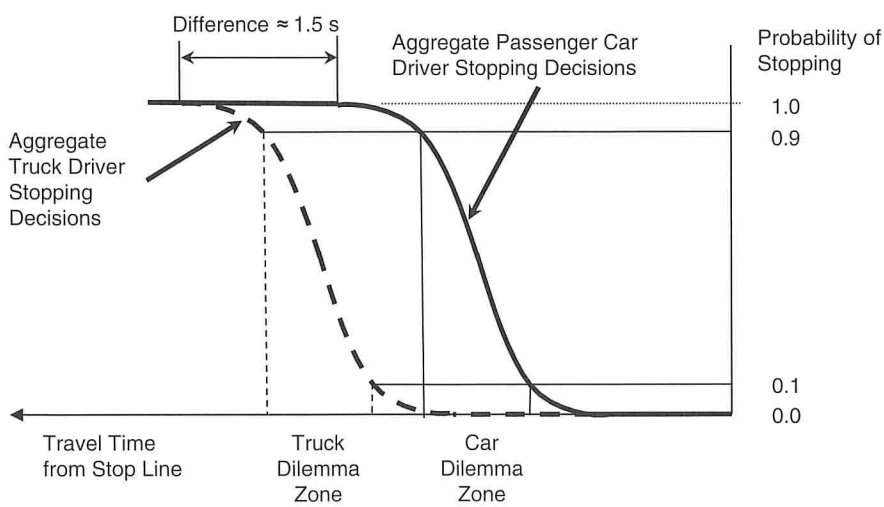


FIGURE 2 Difference between car and truck dilemma zones (*7–9*).

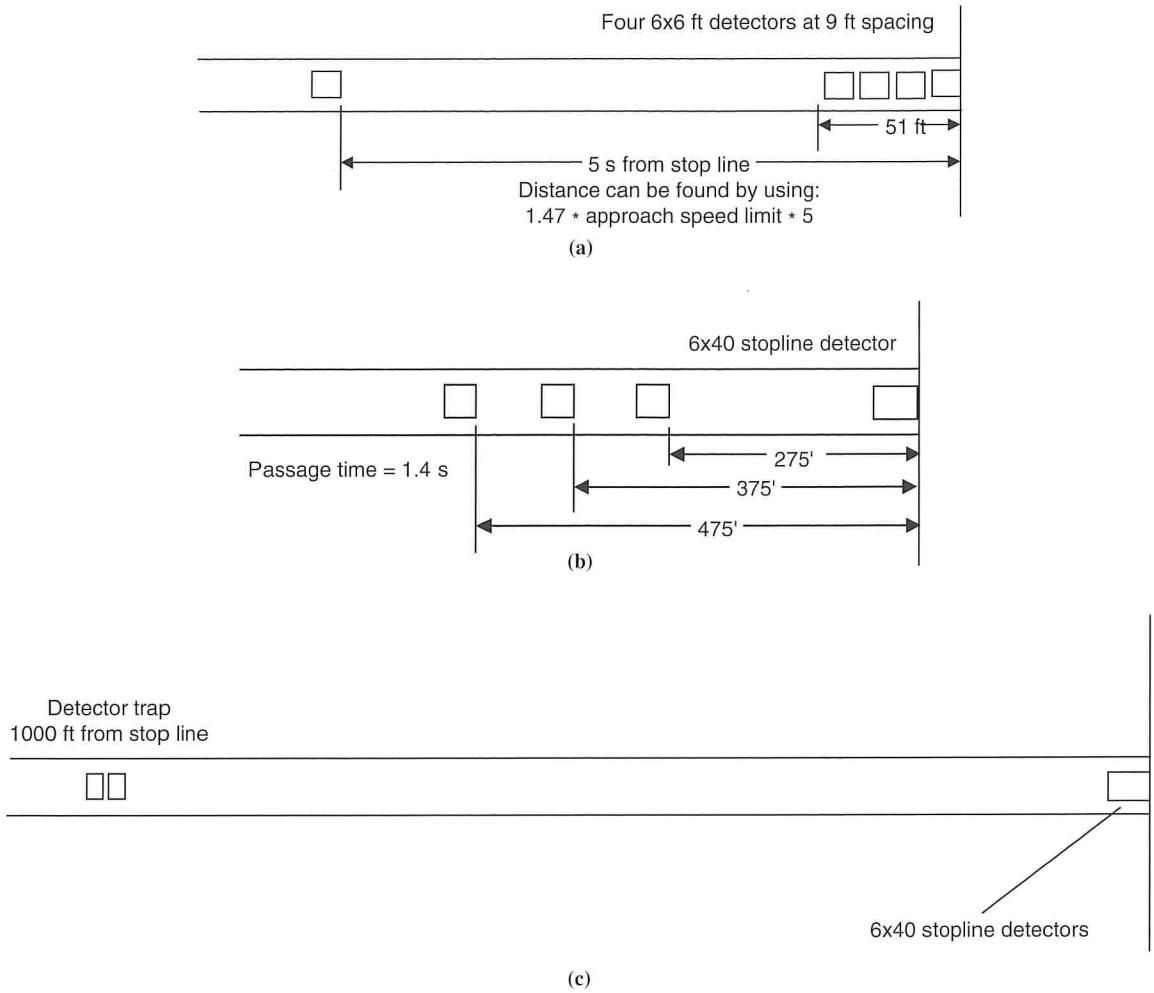


FIGURE 3 Examples of dilemma zone protection designs: (*a*) single-loop detection system (*9*), (*b*) multiple-loop detection system (*6*), and (*c*) detection-control system (*11*).

PROJECT GOALS

The goal of this project was to develop a variety of new dilemma zone protection systems with several levels of strength and costs to better meet the needs of state and local departments of transportation (DOTs). For example, high-end systems will meet the needs of complex, large high-speed intersections, and lower-cost systems will meet the needs of massive deployment by the DOTs. At present, any dilemma zone warning system must include some form of visible warning for drivers somewhere on the roadway or roadside, because in-vehicle warning is not yet practical on a wide scale. In addition, it would be desirable for the new system to incorporate potential future expansion for onboard equipment and vehicle–infrastructure communication for better focus of warnings for individual drivers and vehicles.

NEW APPROACH TO DILEMMA ZONE PROTECTION AND WARNING: SYSTEM DESIGN

The approach taken by the research team was to incorporate the most important features of D-CS and AWEGS into a new system that would provide both dilemma zone protection and warning. The basic layout of the new system, dubbed the detection, control, and warning system (DCWS), is shown in Figure 4. The control system would be similar to D-CS, with some additional modifications, which are described later. The warning system would use in-pavement LED lights to progressively warn each vehicle as needed if that vehicle would be in the dilemma zone at the end of the green signal. D-CS was already predicting vehicle speed and location, so the D-CS vehicle information could be used to provide individualized warning for each vehicle as it approached the intersection. In-pavement devices were chosen to attempt to maximize visibility for the driver being warned while minimizing spillover of the warning to other traffic. Human factors studies are currently being conducted to determine the efficacy of such a system, in particular whether drivers understand the warnings being provided and how they react to them. However, if an in-pavement warning proved to be impractical or ineffective, the warning could be provided by more traditional roadside signage, albeit with a loss of individualized, lane-specific warnings. The same type of warning could be easily incorporated into onboard equipment when this type of equipment becomes more common in vehicles and infrastructure. All of these possible warning configurations are being investigated in human factors studies (*17*).

The modifications to D-CS to create DCWS were fourfold. First, D-CS used hardwired detector loops placed 1,000 ft from the inter-

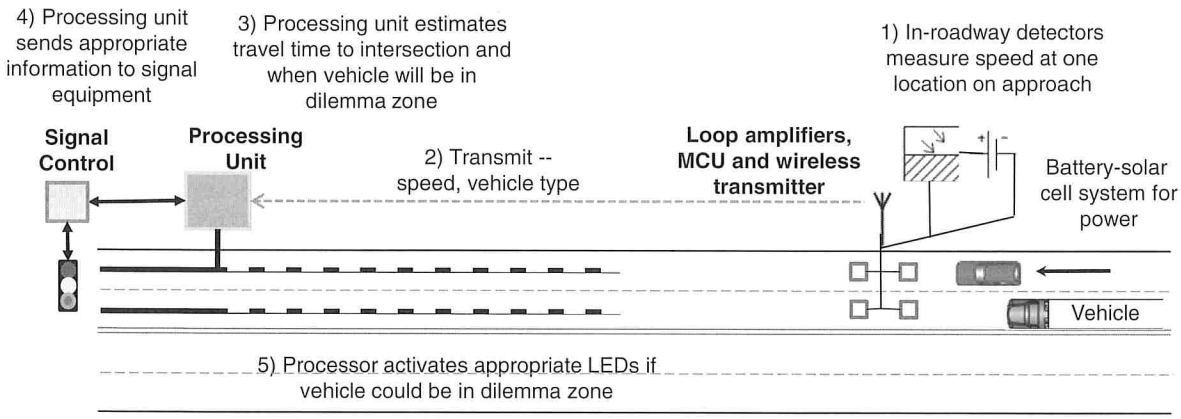


FIGURE 4 DCWS design (MCU = microcontroller unit).

section to determine vehicle speed and length. Trenching, boring, and cabling costs made the installation of these detectors expensive. DCWS would use an upstream microcontroller to calculate vehicle speed and arrival time from the upstream detectors (shown in Figure 4 as inductive loops but that could be any reliable technology), with the microcontroller relaying the data to the processor at the intersection wirelessly. This modification eliminated the cost of cable installation.

Second, D-CS used vehicle length as a surrogate for vehicle type in a variety of calculations, including dilemma zone occupancy. DCWS uses a vehicle-type index instead (i.e., 1 = car, 2 = truck), allowing for faster processing while still accomplishing the goal of designating vehicles by type.

Third, DCWS would incorporate a differential dilemma zone for cars and trucks, as developed previously for D-CS and as illustrated in Figure 2. The dilemma zone for trucks would begin 7.0 s from the intersection (as opposed to 5.5 s for cars) and end 2.5 s from the intersection to reduce the risk of truck-induced rear-end collisions.

Finally, DCWS would use a slightly modified form of the D-CS intelligent phase end (called Stage 2) both to further reduce the frequency of max-out and to provide the warning portion of the system with advance notice that the phase would be ending during Stage 2.

The warning system would piggyback off of the control system, using the same information. In this way the warning system could theoretically be deployed with a more traditional dilemma zone detection scheme and still be fully functional.

TESTING OF HARDWARE-IN-THE-LOOP SIMULATION

Real-time hardware-in-the-loop simulation was used to run DCWS in synchronization with controller hardware. The simulation had two principal objectives:

- To test the modifications to DCWS and
- To provide more information about the frequency of warning events and warning distances.

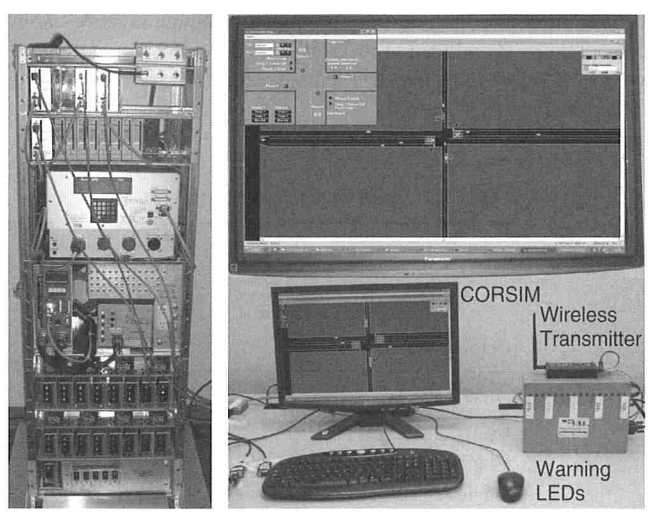
Each of these principal objectives is discussed in the following sections.

Setup of Simulation Test

Real-time simulation testing was performed with the equipment shown in Figure 5. The full cabinet was chosen to ensure that all of the controller and cabinet functions were executed as they would be in the field, without emulation or other approximations. D-CS was originally designed to operate with the National Electrical Manufacturers Association controller family, so that environment was retained for this testing.

Testing of DCWS Algorithm

After the modifications to the D-CS algorithm were made, they were tested to make sure that the system was performing as expected. Real-time simulation runs were performed to test the new D-CS algorithm. Each simulation run required an hour of real time and represented a two-lane highway with an operating speed of 60 miles



(a) (b)

FIGURE 5 DCWS simulation test setup: (*a*) TS-2 controller and (*b*) large-screen display.

per hour (mph), with one of two volume levels and one of four truck percentages:

- Volume levels of 400 or 800 vehicles per hour per lane (vphpl) and
- Truck percentages of 0%, 10%, 20%, 30%, or 40%.

The entire range of conditions required 10 simulation runs. This range was replicated four times, yielding 40 h of simulation. In addition, to ensure that DCWS was still handling Stage 2 and max-out situations appropriately, another set of 10 simulation runs was made with a change to the dilemma zone limits to ensure that max-out would occur more frequently. The dilemma zone limits were increased from the normal 5.5-s to 2.5-s limits to 5.5-s to 0-s limits (i.e., to the stop line), which in turn made max-out much more likely and exercised the Stage 2 and max-out portions of the code more thoroughly. A total of 50 h of real-time simulation was conducted.

For comparison purposes, additional simulation runs were performed with the two more conventional dilemma zone protection systems. The same volume levels and truck percentages were used for each design as for DCWS, allowing a direct comparison between the results of the three designs. To ensure that an equal time was being compared, each set of 10 runs for the conventional designs was replicated five times, yielding 50 h of simulation for each design.

The multiple-loop detection design was used, with a passage time of 1.4 s. The single-loop detection design was also used, with a passage time of 5.0 s. The stop line detectors were allowed to remain active throughout the phase for both designs. From a timing standpoint, allowing stop line detectors to remain active throughout the phase is not desirable. However, it is common practice in many jurisdictions to use a single detector amplifier channel for both the stop line and advance detectors, making it impossible to deactivate the stop line detector after queue clearance.

To avoid the problem with wasted gap time and to reduce cost on detector maintenance at the stop line, some jurisdictions omit the stop line detectors entirely on high-speed intersection approaches. This practice can create completely different timing problems—premature phase ends and an inability to sense the end of stopped queues—that surprise drivers and may not allow stopped queues to clear each cycle. Volume-density operation is one solution that can be used to reduce both of those problems, although practice varies considerably from location to location and a poorly implemented volume-density timing plan can actually make these problems worse rather than better.

PERFORMANCE OF DCWS

In addition to testing the DCWS algorithm, simulation could provide information about the frequency of warning events. The desired information about warning events was as follows:

- The number and percentage of vehicles that required warning,
- The number of warning events per hour, and
- An estimate of warning distances for LED placement.

Each of these three pieces of information is described in greater detail in the following subsections. First, however, some basic signal operating information is needed to place the results in context.

Basic Signal Operations

Table 1 shows some of the basic signal and dilemma zone operating outputs for all of the simulation runs. Intersection delay was used to determine whether the intersection operations remained similar across all of the detection designs. It was recognized that the intersection delay would underestimate delay for minor movements because the majority of traffic at these intersections was through traffic on the major street with little or no delay. The overall intersection delay is similar for all three of the detector configurations; DCWS tends to have somewhat higher delay at high volumes and somewhat lower delay at low volumes. Overall, the difference in intersection delay is fairly small.

The number of vehicles in the dilemma zone is expressed in three ways: as a total number, as the number per 1,000 entering vehicles, and as the number per 100 signal cycles. The overall number of vehicles in the dilemma zone depends on both traffic volume and the number of signal cycles, so as the cycle length decreases the number of vehicles in the dilemma zone can increase because of the increased exposure to the end of the green indication. Normalizing the number of vehicles in the dilemma zone by either volume or number of cycles allows for better comparisons between signals with differing volumes and cycle lengths. Finally, the number of vehicles in the dilemma zone caused by max-out is also shown, with the percentage caused by max-out relative to the total number of vehicles in the dilemma zone also provided for reference. There is a considerable difference between DCWS and the other detection designs in terms of the number of vehicles in the dilemma zone. This difference translates directly into the number of vehicles in the traffic stream that required warning.

TABLE 1 Basic Signal Operations

		Number of Vehicles in the Dilemma Zone				
Detection Design	Intersection Delay (s/veh)	Total	Per 1,000 Entering Vehicles	Per 100 Signal Cycles	Due to Max-Out	Percentage of Total
DCWS, low volume	11.21	20	3.09	0.85	0	0
DCWS, high volume	18.04	188	7.18	13.89	78	41.49
Single-loop design, low volume	15.51	342	18.78	12.74	257	75.15
Single-loop design, high volume	16.29	1,093	30.17	51.96	1,088	99.54
Multiple-loop design, low volume	12.48	158	8.54	4.57	0	0
Multiple-loop design, high volume	14.68	661	18.24	25.63	334	50.68

NOTE: s/veh = seconds per vehicle.

Number and Percentage of Vehicles That Required Warning

The frequency of warning is also important. Warnings that are required too frequently can indicate a dangerous issue that must be addressed or that can cause drivers to ignore the warnings. The simulation results are summarized in Table 2 for DCWS. The percentage of vehicles that required warning was considerably less than 1%. For all 50 h combined, only 0.347% of vehicles required warning (208 out of 60,000). Of the vehicles that required warning, 69 of them (33%) occurred in the final 10 h after the DCWS dilemma zone was adjusted to provide more max-out situations. In the final 10 h, 0.575% of vehicles required warning; in the other 40 h, only 0.150% of vehicles required warning. Most of the vehicles in the last 10 h that required warning needed the warning as a result of max-out (59 of 69). In contrast, only 21 vehicles needed warning as a result of max-out during the first 40 h. Because both detector settings and volume levels affect max-out, passage times and dilemma zone sizes should be set as small as practical for high-speed signalized intersections to avoid max-out.

Table 2 also illustrates a difference in warning events when volume was taken into account. Of the 208 vehicles that required warning, only 20 of them occurred at the low-volume level, 0.1% of the 20,000 vehicles passing through the signal at low volume. The remaining 188 vehicles that required warning (0.470% of 40,000 vehicles) occurred at the higher-volume level. Therefore, an increase in warning events can be expected as volumes increase.

However, warning events are dependent on both volume and cycle length, and a change in one or both may affect their number. At an actuated signal, the actual cycle length varies by demand. Under high-volume conditions on all approaches, each signal's phase tends to max out; cycle lengths are correspondingly long. Also, more vehicles are present that may require warning. Under low-volume conditions, however, cycle lengths may also be long—potentially longer than during high-volume conditions. If traffic demand for minor movements is very light, a traffic signal may dwell in one phase for an extended period of time, resulting in observed cycle lengths of 10 min or more. Consistently short cycles are difficult to achieve with an isolated actuated signal unless recalls are used. Few vehicles are present that would require warning, and because gaps in the traffic stream are readily present, even fewer vehicles will be in the dilemma zone.

One interesting factor relating to signal timing is part of signal timing philosophy. Many jurisdictions advocate what is termed "snappy" operation at traffic signals (i.e., short passage times for detectors to allow phases to change faster) to reduce overall intersection delays. Snappy operation tends to have shorter cycle lengths, which means shorter green phases for all traffic. Shorter cycles mean the ending of more green phases per unit time (say, per hour), which means more requirements for warning vehicles that approach a signalized intersection at high speed. (Snappy operation also means that more vehicles will be stopped on the major road to allow service for minor-movement vehicles.) Therefore, snappy operation will tend to produce more warning events because of higher frequency of the end of the green phase. In contrast, what is termed "sluggish" operation reduces exposure to the end of the green phase but tends to produce max-outs, which have their own negative safety implications. Good operational practice balances the needs of all of the drivers at the intersection, with delay as only one factor to be considered.

Table 3 shows some of the implications of using other detection designs with the warning system. Because of the nature of these other designs, the number of vehicles that require warning is much greater, especially for the single-loop design. Part of the reason for the large difference in the number of vehicles that require warning is the somewhat unpredictable nature of phase ends for these designs, and part of the reason is the result of the relative sluggishness of these designs compared with that of DCWS. In particular, the single-loop design requires a very long (5 s) passage time for the upstream detector to allow an arriving vehicle to reach the intersection before the phase changes. In many cases, if a stop line detector is present, it will have the same passage time as the upstream detector, effectively requiring a gap of approximately 10 s before the green phase can end.

TABLE 2 Vehicles Requiring Warning: DCWS Only

Simulation Runs	Simulated Vehicles	Vehicles Requiring Warning	Percent
All 50 h	60,000	208	0.347
Max-out only		78	
Stage 2 only		61	
Low volume (400 vphpl)	20,000	20	0.100
High volume (800 vphpl)	40,000	188	0.470
First 40 h	48,000	72	0.150
Max-out only		21	
Stage 2 only		51	
Low volume (400 vphpl)	16,000	15	0.094
High volume (800 vphpl)	32,000	124	0.388
Last 10 h	12,000	69	0.575
Max-out only		59	
Stage 2 only		10	
Low volume (400 vphpl)	4,000	5	0.125
High volume (800 vphpl)	8,000	64	0.800

NOTE: Shading = not possible to determine number of simulated vehicles contributing to vehicles that required warning.

TABLE 3 Vehicles Requiring Warning: All Systems

Simulation Runs	Simulated Vehicles	Vehicles Requiring Warning	Percent
DCWS, all 50 h	60,000	208	0.347
Max-out only		78	
Stage 2 only		61	
Low volume (400 vphpl)	20,000	20	0.100
High volume (800 vphpl)	40,000	188	0.470
Single-loop detection design	60,000	1,425	2.375
Max-out only		1,335	
Stage 2 only		—	
Low volume (400 vphpl)	20,000	342	1.710
High volume (800 vphpl)	40,000	1,093	2.733
Multiple-loop detection design	60,000	819	1.365
Max-out only		335	
Stage 2 only		—	
Low volume (400 vphpl)	20,000	158	0.790
High volume (800 vphpl)	40,000	661	1.653

NOTE: Shading = not possible to determine number of simulated vehicles contributing to vehicles that required warning; — = detection design does not have Stage 2 period.

Gaps of this size are rare at high volumes, so the phase tends to max out frequently. Of the 1,425 vehicles that required warning with the single-loop detection design, 93.7% (1,335) were due to max-out. At high volume, the proportion of vehicles that required warning as a result of max-out was essentially 100%. If a single-loop detection design is used and volumes are expected to be high, it would make no difference whether the warning system is a static sign with flashers and a preprogrammed 3-s flash period or the more sophisticated warning system proposed here.

In contrast, the multiple-loop design uses a passage time of only 1.4 s. No max-outs occurred during the low-volume runs when the multiple-loop detection design was used. All 335 vehicles in the dilemma zone as a result of max-out got there during the high-volume runs.

When comparing the results for DCWS and the other detection designs in Table 3, remember that the final 10 hours of simulation in Table 2 used a much larger truck dilemma zone. In a comparison of the first 40 hours of DCWS simulation with the results shown in Table 3, DCWS reduced the percentage of vehicles that required warnings by a factor of 5 compared with the multiple-loop detection design and a factor of 8 compared with the single-loop detection design. DCWS is clearly effective at reducing the number of vehicles in the dilemma zone, which in turn reduces the number of warnings that would be necessary for this system compared with a warning system associated with other detection designs.

Number of Warning Events per Hour

Proportionally few vehicles in the traffic stream require warning when they approach an intersection, so the small numbers produced are difficult to interpret. A more revealing way to look at the frequency of warning events is to show the number of events per hour rather than the percentage of vehicles warned. Table 3 shows an example of how warning events would be influenced by traffic volume on single-lane approaches. For low-volume situations, very few if any warnings would be needed. Very little traffic would be present, and cycle lengths tend to be long as a result of dwell time, minimizing the number of phase ends that occur. When a green phase does need to end, there are frequent, adequate gaps to allow the phase to end without vehicles being present in the dilemma zone. Because there is randomness in the traffic stream, on occasion a vehicle may be in the dilemma zone at the end of the green phase when volumes are low, but the frequency is very low.

As volumes increase, the number of warnings increases because there is more demand for minor movements and there are more vehicles present on the major movements. As volumes increase, the number of opportunities (phase ends) per hour will actually decrease but the number of vehicles on the major movement will increase, counteracting the reduction in the number of opportunities. Also, the need for warning is not consistent: some phases will have heavy traffic at the end of the phase; some will have none because random platooning occurs in the traffic stream.

At high volumes, the number of max-outs increases, eventually reaching 100% (every cycle). A max-out effectively ensures that a phase will end with at least one vehicle in the dilemma zone, so virtually every phase end would require warning at least one vehicle. However, the number of opportunities (phase ends) becomes effectively constant, which creates an upper limit on the number of warning events per hour.

Table 4 also shows the number of warning events per hour that could be expected when either a multiple-loop or a single-loop detection design is used. These values are similar to the values shown for DCWS at high volumes, indicating that max-out is a common issue at those volumes. However, DCWS has significantly fewer warning events at lower volumes.

TABLE 4 Number of Warning Events per Hour

Detection Design	Volume Level	Approximate Volume (vphpl)	Warning Events per Hour per Lane
DCWS	Low	400	1.94
	High	800	16.34
Multiple-loop	Low	400	6.61
	High	800	17.70
Single-loop	Low	400	6.86
	High	800	22.17

ESTIMATE OF WARNING DISTANCES FOR LED PLACEMENT

With the simulation results, it was possible to estimate the visibility ranges that would be required for the in-pavement warning system. Each end of the system is located with slightly different criteria. For the upstream end of the warning system, the LEDs must be visible at a point 3 s before the dilemma zone. This extra time will allow for a warning before the actual phase change occurs. The maximum upstream distance a vehicle would be located that would need warning was approximately 800 ft.

The downstream end of the warning system has to correspond with the downstream end of the dilemma zone, or 2.5 s from the intersection. At the lowest "high" speed typically applied to signalized intersections (45 mph), the minimum distance from the intersection would be 165 ft. Therefore, the range of visibility for the warning system would be approximately from 150 ft to 800 ft from the intersection.

The range of distances mentioned is for the visibility of the warning system only, not for the actual placement of LEDs. The LEDs may be visible from 100 to 200 ft away, depending on the number of LEDs in the cluster, LED directionality, conspicuity, and other factors, so they will not have to be placed as far as 800 ft from the intersection. The LEDs will also be visible over some distance, so the final LED will not necessarily have to be placed near or at the intersection stop line itself. At present, the visibility cone for the LEDs is not known; the exact placement locations cannot be determined at this time. Yellow LEDs would be used, consistent with their purpose of warning drivers, and the flash rate would probably be in the range of 50 to 60 flashes per minute, similar to existing systems. The human factors study in Phase II will help determine the exact placement of LEDs for maximum effect.

Currently, the U.S. Department of Transportation is testing a variety of connect vehicle technologies for improving intersection safety that are capable of addressing dilemma zone issues. A possible area of future study is the addition to the infrastructure-based dilemma zone protection system developed in this project of a connected vehicle support module that feeds the connected vehicles through 5.9-GHz wireless communication. This module will take vehicle information from the onboard equipment (OBE) and feed the advisory information to connected vehicles via roadside equipment. The information from OBE will help the operations of the system to better serve the vehicles without OBEs.

CONCLUSIONS AND FUTURE DEVELOPMENT

The new system, DCWS, which is based on a combination of a proven signal control system and a new warning system, can both reduce the number of vehicles in the dilemma zone by direct control of the signal phases and warn drivers who could not be kept out of the dilemma zone that the signal indication is about to change and that they should probably stop. Other systems do one or the other but not both. The inclusion of a warning system in DCWS provides valuable, real-time information to help a driver in making a correct decision to stop or proceed. The new system has the capacity to provide real-time warning via LED flashers, controlled by a microprocessing board and coordinated with a signal controller, which provide drivers with information necessary to make safer decisions. Theoretically, the warning system can be decoupled from the intersection control system, making it possible to provide warning for intersections with more traditional types of dilemma zone protection; however, the simulation results indicate that more warnings would be necessary with those systems than with DCWS. The advent and development of OBE technologies may aid in the future development and use of warning systems for dilemma zones, making high-speed intersections safer and easier for drivers to navigate.

Future development includes a human factors investigation of different types of warning configurations, including in-pavement warning lights and roadside devices, to find the most effective system. This investigation is currently underway (*17*). In addition, further development of the DCWS algorithm is continuing and includes enhancing system accuracy and reliability with more detection, distinguishing between through and turning vehicles, providing warning for situations involving preemption and coordination, and providing both control and warning for major–major high-speed intersections (i.e., where two major streets intersect). A demonstration of the system was implemented at the test intersection of the Turner–Fairbank Highway Research Center in spring of 2012.

ACKNOWLEDGMENT

Work presented in this paper was supported by a U.S. DOT Phase I Small Business Innovation Research contract.

REFERENCES

1. Gazis, D., R. Herman, and A. Maradudin. The Problem of the Amber Signal Light in Traffic Flow. *Traffic Engineering,* Vol. 30, No. 7, 1960, pp. 19–26.
2. Technical Committee 18, Southern Section, Institute of Transportation Engineers. Small Area Detection at Intersection Approaches. *Traffic Engineering,* Vol. 44, No. 2, 1974, pp. 8–17.
3. *Traffic Engineering Handbook,* 4th ed. Institute of Transportation Engineers, Washington, D.C., 1999.
4. Zegeer, C. V., and R. C. Deen. Green Extension System at High-Speed Intersections. *ITE Journal,* Vol. 48, No. 11, 1978, pp. 19–24.
5. Bonneson, J. A., P. T. McCoy, and B. A. Moen. *Traffic Detector Design and Evaluation Guidelines.* Report TRP-02-31-93. Nebraska Department of Roads, Lincoln, 1993.
6. Middleton, D., R. L. Nowlin, M. Shafer, A. H. Parham, and D. Jasek. *Evaluation of Detector Placement for High-Speed Approaches to Signalized Intersections.* Report TX-98/3977-1. Texas Department of Transportation, Austin, 1997.
7. Gates, T. J., D. A. Noyce, L. Laracuente, and E. V. Nordheim. Analysis of Driver Behavior in Dilemma Zones at Signalized Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2030,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 29–39.
8. Zimmerman, K. H. Additional Dilemma Zone Protection for Trucks at High-Speed Signalized Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2009,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 82–88.
9. Gates, T. J., and D. A. Noyce. Dilemma Zone Driver Behavior as a Function of Vehicle Type, Time of Day, and Platooning. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2149,* Transportation Research Board of the National Academies, Washington, D.C., 2010, pp. 84–93.
10. Indiana Department of Transportation. *Indiana Design Manual.* Indianapolis, 2011. http://www.in.gov/dot/div/contracts/standards/dm/2011/index.html.
11. Bonneson, J. A., D. A. Middleton, K. H. Zimmerman, H. A. Charara, and M. Abbas. *Intelligent Detection-Control System for Rural Signalized Intersections.* Report FHWA/TX-03/4022-2. Texas Transportation Institute, College Station, 2002.
12. *Manual on Uniform Traffic Control Devices.* FHWA, U.S. Department of Transportation, 2003.
13. McCoy, P. T., and G. Pesti. Improving Dilemma-Zone Protection of Advance Detection with Advance-Warning Flashers. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1844,* Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 11–17.
14. Huang, X. P., and P. D. Pant. Simulation-Neural Network Model for Evaluating Dilemma Zone Problems at High-Speed Signalized Intersections. In *Transportation Research Record 1456,* TRB, National Research Council, Washington, D.C., 1994, pp. 34–42.
15. Pant, P. D., and X. H. Huang. Active Advance Warning Signs at High-Speed Signalized Intersections: Results of a Study in Ohio. In *Transportation Research Record 1368,* TRB, National Research Council, Washington, D.C., 1992, pp. 18–26.
16. Sunkari, S. R., H. A. Caharara, R. T. Parker, and T. A. Palekar. *Enhancements to the Advance Warning of End of Green System (AWEGS).* Report FHWA/TX-07/0-5113-1. Texas Transportation Institute, College Station, 2006.
17. McIntyre, S., L. Gugerty, D. Link, K. Zimmerman, D. Tolani, R. Pokorny, and P. Huang. A Human Factors Study of Dilemma Zone Warning Systems. Presented at 91st Annual Meeting of the Transportation Research Board, Washington, D.C., 2012.

The Traffic Control Devices Committee peer-reviewed this paper.

Evaluation of Two Treatments for Reducing Crashes Related to Traffic Signal Change Intervals

Bhagwant Persaud, Frank Gross, and Raghavan Srinivasan

Two related evaluation studies that were conducted under NCHRP Project 17-35 are presented. The research objective was to use the most appropriate analytical methods and the best available data to build on previous research in developing crash modification factors (CMFs) for two treatments for reducing crashes related to traffic signal change intervals: modifying the change interval and installing dynamic signal warning flashers. Three evaluation methods were used as appropriate—the empirical Bayes before–after method, the comparison group before–after method, and cross-sectional multiple regression models. A secondary objective of using cross-sectional models for some evaluations was to examine the comparability of before–after and cross-sectional studies, a subject of topical interest in CMF development. There was a general safety benefit to installing dynamic signal warning flashers, with indications that crash reductions could be obtained overall and for several crash types, including injury, angle, and heavy-vehicle crashes. For the change-interval modification, the before–after study results showed significant reductions (at the 5% level) in total, injury, and rear-end crashes under various scenarios. For both treatments, the results from the cross-sectional analysis were relatively consistent with those from the before–after analysis.

NCHRP Report 500, *Volume 12: A Guide for Reducing Collisions at Signalized Intersections,* identifies safety issues related to signalized intersections as well as potential countermeasures to address the safety issues (*1*). Several strategies are identified to reduce the frequency and severity of intersection conflicts through traffic control and operational improvements. A specific strategy listed in this volume is to "optimize clearance intervals." Strategies to improve driver awareness of intersections and signal control are also identified. Another specific strategy listed in this volume is to "improve signing and delineation," which includes dynamic signal warning flashers (DSWFs). This paper summarizes the safety evaluation of these two treatments, which both pertain to crashes related to the traffic signal change interval: modifying the change interval and installing DSWFs. The project report contains considerably more details (*2*).

B. Persaud, Department of Civil Engineering, Ryerson University, 350 Victoria Street, Toronto, Ontario M5B 2K3, Canada. F. Gross, Vanasse Hangen Brustlin, Inc., 333 Fayetteville Street, Suite 1450, Raleigh, NC 27601. R. Srinivasan, Highway Safety Research Center, University of North Carolina, CB 3430, 730 Martin Luther King Jr. Boulevard, Chapel Hill, NC 27599-3430. Corresponding author: B. Persaud, bpersaud@ryerson.ca.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 38–45.
DOI: 10.3141/2298-05

The change interval is the time allocated for the yellow and all-red phases for a given approach at a signalized intersection. Ideally, the yellow phase should be such that vehicles within the safe stopping distance of the stop bar should be able to enter the intersection legally during the yellow phase when they are traveling at the approach speed. Thus, inadequate yellow and all-red phases can lead to safety issues. For example, yellow phases that are too short may create a dilemma zone, for which the phase change interval is such that a vehicle in this section of roadway is too far away to pass safely through the intersection and too close to stop before the intersection. Yellow phases that are too long may result in inconsistent driver behavior (e.g., some drivers may stop during the yellow phase while other drivers try to beat the light), which can result in rear-end crashes. Thus, if an appropriate change interval can be identified and the signal timing can be modified accordingly, there is the potential to reduce the frequency and severity of crashes at signalized intersections.

DSWFs provide drivers with advance notice of the phase change. Specifically, the DSWFs are linked to the signal and are actuated when the signal is about to change from green to yellow. The flashers are typically located several hundred feet in advance of the intersection, either along the roadside or overhead, and are actuated at a time when the driver would not be able to clear the intersection before the onset of the red phase. Thus, DSWF placement is related to several factors, including the 85th percentile speed, driver reaction time, coefficient of friction, grade, and length of the change interval. The distance may be measured from the stop bar, but if the sight distance to the signal is limited and the intersection experiences lengthy queues, it may be desirable to measure the distance based on the end of the queue and not the stop bar. By enhancing awareness of the impending red phase, DSWFs should reduce conflicts related to sight-distance issues or perception–reaction time.

Reductions in the number of crashes have been shown to be associated with modifications to the change interval. Specifically, past studies have shown favorable safety effects when the change interval is modified to better match the yellow and all-red times based on the ITE procedure (*3*). For example, Retting et al. evaluated 40 sites requiring increases in the duration of change-interval timings based on ITE guidelines compared with 56 control sites (*4*). The study results showed a statistically significant 12% reduction in all injury crashes at the experimental sites compared with the control sites. The same study showed an insignificant effect on angle and rear-end crashes but noted a 37% reduction in crashes involving pedestrians or bicyclists.

There have been several evaluations of DSWFs over the past three decades and they have used a variety of measures and designs.

Some of the more recent studies are by Sayed et al. (*5*), Radalj (*6*), and others (*7, 8*). Some evaluations were crash-based studies; others examined speed or conflict data (e.g., instances of red light running). Among the crash-based studies, the following designs have been used: naïve before–after, before–after with crash rates, before–after with control group, and cross-sectional regression models. The results of these various studies have been diverse: many suggested positive safety effects but others suggested negligible or negative effects.

The objective of the two evaluation studies discussed here, conducted under NCHRP Project 17-35, was to use the most appropriate analytical methods and the best available data to build on previous research in developing crash modification factors (CMFs) for the two treatments (*2*). Of particular interest was the potential variation of the CMFs with circumstances of implementation.

METHODOLOGY

Three evaluation methods were used in this study. The primary, and preferred, one is the empirical Bayes (EB) before–after method, which is considered to be one of the best methods for conducting before–after studies in that it properly accounts for regression to the mean. The second method is the comparison group before–after method. This method does not effectively address the bias caused by regression to the mean, but it is effective in accounting for other nontreatment effects such as those caused by trends in crash reporting and changes in traffic volume. The third method is based on cross-sectional multiple regression models in which the CMFs are derived from coefficients of the respective variables. A secondary objective of using cross-sectional models for some evaluations was to examine the comparability of before–after and cross-sectional studies. This comparability is a subject of topical interest in CMF development for which there is little research. More details on these methods can be found in a recent FHWA guide (*9*).

Before–After Methods

The before–after methods estimate the number of expected crashes that would have occurred in the after period (λ) and compare that number with the number of reported crashes in the after period (π). In accordance with Hauer (*10*), the following steps are used to estimate λ:

1. Identify a reference group of untreated sites that is otherwise similar to the treatment group.
2. Use the reference–comparison group data to estimate safety performance functions (SPFs), which are mathematical equations, that predict the number of crashes of different types as a function of traffic volumes and other site characteristics. Typically, SPFs are predictive rather than causal models; generalized linear modeling with a negative binomial error structure is used for their estimation.
3. In estimating SPFs, calibrate annual SPF multipliers (time trend factors) to account for the temporal effects (e.g., variation in weather, demography, and crash reporting) on safety.
4. Use the SPFs, the annual SPF multipliers, and data on traffic volumes and site characteristics for each year in the before period for each treatment site to estimate the number of crashes that would be predicted in each year of the before period and after periods for each treatment site.
5. Estimate λ (expected crashes in the after period if the treatment had not been implemented) as the product of the expected number of crashes in the before period and the sum of the annual SPF predictions for the after period divided by the sum of these predictions for the before period (for each treatment site). For the comparison group before–after method, the observed crash count in the before period is used to estimate the expected number of crashes in the before period. This estimate could be biased if crashes are selected for treatment because of a randomly high, observed crash count. The EB method adjusts for this possible regression to the mean bias because it uses the predicted number of crashes in the before period (from the SPFs) and the observed crashes in the before period to estimate the EB-expected number of crashes in the before period in each treatment site. Both methods also estimate the variance of λ.
6. The estimate of λ is then summed over all sites in a treatment group of interest and compared with the count of crashes during the after period in that group. The variance of λ is also summed over all sites in the strategy group.
7. These parameters (the summation of λ and its variance) are then used, along with the summation of crash counts after treatment, to estimate an effect of the treatment (CMF). The standard deviation of the CMF is also estimated, which makes it possible to determine if the CMF is statistically different from 1.0 for a specific level of significance.

Cross-Sectional Regression Models

Cross-sectional studies derive CMFs by comparing the crash statistics from sites with and without the treatment. If it is possible to find sites that are similar to each other (apart from having or not having the treatment), then the CMF could be defined as the ratio of the average number of crashes at sites with the treatment to the average number of crashes at the sites without the treatment. In practice it is difficult to find sites that are similar to each other, so regression models are typically used. The state of the art is to use negative binomial regression models in which crash frequency is the dependent variable and the independent variables may include site characteristics such as major- and minor-road annual average daily traffic (AADT). The model coefficients are used to derive the CMFs, which makes such models fundamentally different from SPFs used in the EB methodology.

At this time, the CMFs from cross-sectional models are not considered as reliable as CMFs derived from well-designed before–after studies, unless they can be corroborated with results from rigorous before–after studies or multiple cross-sectional studies (*11*). Common concerns related to cross-sectional analyses include

- Misspecification of model functional form,
- Confounding effects,
- Interaction effects, and
- Inconsistency among results from different studies of the same treatment.

These concerns are not as critical for the predictive SPFs used in the EB methodology; those models, unlike the cross-sectional regression models, are not intended to infer causation. The following techniques were used in this study to address these common concerns:

- Several functional forms were explored to test the sensitivity of the coefficients.
- Several covariates were considered for inclusion in the model.
- Interaction effects were tested and included as necessary.

EVALUATION SUMMARY: DSWF

Data Used

Treatment sites where DSWFs had been installed were identified in Nevada, North Carolina, and Virginia, and geometric, traffic volume, and crash data were assembled. Data from 1994 to 2008 were available for Nevada, but only a data subset was used, to avoid any major construction activity. For North Carolina, data from 1993 to 2009 were used. Data from 1998 to 2008 were available for Virginia, but again only a data subset was used to avoid construction activity.

Methodology

With respect to all the treatment sites in Nevada and most of the treatment sites in Virginia, it was discovered that the traffic signals and DSWFs were installed at the same time. This observation had an important impact on the selection of an analytical method for this analysis. Because it would be difficult to separate the effects of the signal installation from the effects of the DSWF installation, the use of before–after methods for those sites would be problematic. In contrast, all of the treatment sites in North Carolina were already signalized when the DSWFs were installed, so a before–after method could be used.

With respect to the Nevada data, a cross-sectional method was used that included two groups of sites: one group consisted of signalized intersections where DSWFs were present, and the other group consisted of signalized intersections where DSWFs were not present. In all, 261 site-years and 3,224 total crashes were included in this analysis.

With respect to the Virginia data, two analytical methods were used. A cross-sectional analysis was conducted with the Virginia data that included two groups of sites: one group of sites consisted of signalized intersections where DSWFs were present and the other group consisted of signalized intersections where DSWFs were not present. The Virginia cross-sectional analysis included 452 site-years and 1,201 total crashes. A before–after method with comparison group was also used with the Virginia data with the goal of validating the results of the cross-sectional analysis. This analysis was possible because, for a subset of the treatment sites in Virginia, the DSWF installations occurred after the traffic signal installations. The Virginia before–after analysis included 67 site-years and 159 crashes in the before period and 83 site-years and 190 crashes in the after period.

Another cross-sectional analysis was performed with a data set that combined the Nevada and Virginia data. As with the individual state analyses, two groups were defined. One group consisted of sites in Nevada or Virginia where DSWFs had been installed, and the other group consisted of sites in Nevada or Virginia where DSWFs had not been installed.

With the North Carolina data, the DSWFs were installed at intersections that were already signalized. Consequently, separating the effects of signal installation and DSWF installation was not an issue, so the EB before–after method could be used. In all, 180 site-years and 1,256 crashes were included in this analysis for the treatment sites. The treatment group contained 14 sites, 1,000 crashes in the before period, and 256 crashes in the after period. The reference group consisted of 63 signalized intersections in North Carolina with 5,948 crashes.

Results for DSWFs

The results of the combined cross-sectional analysis for Nevada and Virginia, with an indicator variable to identify the state, were ultimately deemed to be the most reliable because they pertained to a multistate database and because they were corroborated, for the most part, by the results of the before–after analyses. The project report by Srinivasan et al. provides details of these analyses (*2*). Various forms of AADT were used to develop preliminary models. The following variables were considered in the development of the combined model:

- Major-road AADT [ln(major AADT) was also considered],
- Minor-road AADT [ln(minor AADT) was also considered],
- Total AADT [ln(total AADT) was also considered],
- Missing minor (indicator for sites where minor-road AADT was unavailable),
- DSWF (1/0 indicator for DSWF),
- State (1/0 indicator for state; 1 = Virginia),
- Approaches (1/0 indicator; 1 = four-legged),
- Speed limit [1/0 indicator; 1 = 55 miles per hour (mph) or greater], and
- Interaction (DSWF × AADT term).

The final models, shown in Table 1, were selected on the basis of several goodness-of-fit-measures. The general functional form of the model assumed for this analysis is shown in Equation 1.

$$\text{crashes/year} = \exp\left(\alpha + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n\right) \quad (1)$$

where α and β_1 to β_n are the parameters estimated in the model calibration process and X_1 to X_n are covariates included in the model.

It was necessary to make an adjustment for the repeated observations (i.e., same site over multiple years). The model used a clustered robust standard error for this adjustment, identifying individual sites as the cluster variable. In evaluating the uncertainty associated with each CMF, a method correction factor (MCF) should be applied to the standard error, as recommended in developing the CMFs in the *Highway Safety Manual* (*12*). The MCF is based on the strength of the study design. Relatively strong study designs are assigned an MCF value that is close to 1.0. An MCF value of 2 is recommended to adjust the results from this analysis. This value is related to regression-based cross-sectional studies that use a conventional functional form and account for several important confounding factors.

Several additional modeling techniques were used to validate the results of the regression analysis. These included variations of the cross-sectional analysis (e.g., different functional forms and subsets of data) and a before–after analysis with comparison group.

The cross-sectional analysis for the two states combined shows a reduction in total crashes at intersections that had DSWFs. The results from the cross-sectional analysis for the individual states, as well as those from the North Carolina EB before–after analysis and the Virginia comparison group before–after analysis, validated these findings. The results in Table 1 also suggest that DSWFs may help to reduce angle, injury, and heavy-vehicle crashes, although the sample size was limited for many of the individual crash types. It was possible to investigate both angle and injury crashes with all three methods; the results consistently indicated a reduction in expected crashes with the presence of DSWFs. The results were less

TABLE 1 Combined Model for Nevada and Virginia for Various Crash Types

Covariate	Coefficient (robust standard error)				
Crash Group	Total	Rear End	Angle	Injury	Heavy Vehicle
Total AADT	0.0000405	−0.0180	−0.115	−0.107	−0.333
	(0.00000392)	(0.178)	(0.181)	(0.167)	(0.299)
Missing minor	−0.0581	−0.0180	−0.1148	−0.1067	−0.3333
	(0.1435)	(0.1776)	(0.1808)	(0.1672)	(0.2991)
DSWF	−0.2057	−0.2326	−0.2946	−0.1984	−0.0450
	(0.1412)	(0.1839)	(0.2143)	(0.1651)	(0.2403)
State (1 = Virginia)	−0.0663	−0.4972	−0.6643	−0.1296	−0.0024
	(0.1673)	(0.1677)	(0.2945)	(0.2207)	(0.3215)
Approaches (1 = four-legged)	−0.0723	−0.2833	0.2820	−0.0687	0.0910
	(0.1450)	(0.1865)	(0.1947)	(0.1613)	(0.2149)
Area type (1 = rural)	−0.1682	−0.3928	0.0713	−0.4599	−0.1845
	(0.2238)	(0.2705)	(0.3126)	(0.2707)	(0.3788)
Speed (1 = ≥55 mph)	0.0809	0.0646	−0.0060	0.2163	0.3403
	(0.1299)	(0.1530)	(0.1965)	(0.1520)	(0.2469)
Constant (α)	0.9945	0.4406	−0.4453	−0.1651	−2.1864
	(0.2878)	(0.3952)	(0.3029)	(0.3237)	(0.4637)
Dispersion parameter	0.3476	0.4872	0.4996	0.3981	0.7827
	(0.0555)	(0.1031)	(0.1030)	(0.0772)	(0.2546)
Log pseudolikelihood	−1,730.8	−1,423.2	−1,020.2	−1,178.7	−524.7
Wald χ^2	266.28	268.14	81.90	150.28	113.69
Prob > χ^2	0.0000	0.0000	0.0000	0.0000	0.0000
CMF	0.814	0.792	0.745	0.820	0.956
Standard error	0.062	0.079	0.086	0.083	0.177
Sample size (crashes)	4,425	2,651	961	1,450	267

consistent for rear-end crashes. The cross-sectional and comparison group analyses were similar, indicating a reduction in expected rear-end crashes with the presence of DSWFs. However, the EB analysis (for North Carolina) indicated an increase in rear-end crashes.

In summary, the results seem to indicate that the DSWFs do provide a benefit with the largest percentage reduction in angle crashes. The relatively large reduction in crashes that result in fatality and injury is likely the greatest benefit of the flashers in terms of overall safety. Future research could investigate the safety effects of the many variations of DSWFs, including roadside and overhead signs.

EVALUATION SUMMARY: INCREASING THE CHANGE INTERVAL

Data Used

Geometric, traffic volume, signal timing, and crash data for both treatment and reference sites were acquired from the states of California (1992 to 2002) and Maryland (1992 to 2002) to facilitate the analysis. Specifically, data were obtained in California from the cities of San Diego and San Francisco and in Maryland from the counties of Howard and Montgomery. The sites provided data for two types of signalized intersections: signalized intersections where the change interval was modified during the study period, and signalized intersections where the change interval was not modified during the study period. If there were major changes to the geometry or operations during the study period, the sites were excluded.

Methodology

The primary analysis methodology used was the EB before–after analysis (previously described). The evaluation analyzed the effects of the treatment on crash frequencies for different crash types and severities before and after the treatment.

Specifically, the EB analysis was used to investigate five specific scenarios. Three scenarios were related to various combinations of increasing the yellow and all-red times: increasing both the yellow and all-red phases, increasing the all-red phase only, and increasing the yellow phase only.

Two other scenarios were investigated that compared the total change interval with the ITE recommended practice. In both cases, the before condition was represented by signalized intersections where the total change interval was smaller than the ITE recommended practice. The after period was represented by signalized intersections with the following characteristics: the total change interval remained smaller than the ITE recommended practice and the total change interval was greater than the ITE recommended practice.

The analyses attempted to develop CMFs by severity (i.e., fatality–injury vs. total crashes) and by crash types (i.e., total, angle, and rear end) for both states. The before–after analysis was based on a total of 31 treatment sites, as noted in Table 2.

Data for signalized intersections, similar to those where the change interval was modified, were sought for use in developing the SPFs required for the EB methodology. Untreated reference sites (i.e., sites where no modifications were made to the change interval during the study period) were identified in each jurisdiction. The

TABLE 2 Number of Sites for Treatment and Reference Groups

Location	Treatment Sites	Reference Sites
Howard County	2	29
Montgomery County	6	38
San Diego	16	36
San Francisco	7	32
Total	31	135

data were used to directly calibrate SPFs for total, injury, rear-end, and angle crashes separately for each jurisdiction. The SPFs used for this evaluation are summarized here.

SPFs for Howard County, Maryland

$$\text{total/year} = \exp(-9.179)(\text{majorADT})^{0.740}(\text{minorADT})^{0.494} \qquad k = 0.482$$

$$\text{injury/year} = \exp(-9.348)(\text{majorADT})^{0.696}(\text{minorADT})^{0.396} \qquad k = 0.685$$

$$\text{rear end/year} = \exp(-14.123)(\text{majorADT})^{1.236}(\text{minorADT})^{0.427} \qquad k = 0.756$$

$$\text{angle/year} = \exp(-7.768)(\text{totalADT})^{0.805} \qquad k = 0.911$$

where

total/year = predicted number of total intersection crashes per year,
injury/year = predicted number of injury-related intersection crashes per year,
rear end/year = predicted number of rear-end intersection crashes per year,
angle/year = predicted number of angle intersection crashes per year,
majorADT = average daily traffic (ADT) on major road,
minorADT = average daily traffic on minor road,
totalADT = total entering volume, and
k = overdispersion parameter of the model.

SPFs for Montgomery County, Maryland

$$\text{total/year} = \exp(-8.431 - 0.646 * \text{protected})(\text{majorADT})^{0.640} * (\text{minorADT})^{0.492} \qquad k = 0.397$$

$$\text{injury/year} = \exp(-9.026 - 0.676 * \text{protected})(\text{majorADT})^{0.583} * (\text{minorADT})^{0.529} \qquad k = 0.502$$

$$\text{rear end/year} = \exp(-8.914 + 0.022 * \text{maxspeed})(\text{majorADT})^{0.707} * (\text{minorADT})^{0.233} \qquad k = 0.458$$

$$\text{angle/year} = \exp(-5.275 + 0.679 * \text{fourleg})(\text{totalADT})^{0.507} \qquad k = 0.656$$

where

protected = indicator for left-turn phasing on major road (1 = fully protected; 0 otherwise),
maxspeed = maximum posted approach speed (mph), and
fourleg = indicator for number of approaches (1 = four-legged; 0 otherwise).

SPFs for San Diego and San Francisco, California

$$\text{total/year} = \exp(-9.028 + 0.225 * \text{SF} - 0.522 * \text{speed40}) * (\text{totalADT})^{1.023} \qquad k = 0.272$$

$$\text{injury/year} = \exp(-9.002 + 0.046 * \text{SF} - 0.527 * \text{speed40}) * (\text{totalADT})^{1.018} \qquad k = 0.281$$

$$\text{rear end/year} = \exp(-11.221 - 0.223 * \text{SF})(\text{majorADT})^{0.680} * (\text{minorADT})^{0.551} \qquad k = 0.228$$

$$\text{angle/year} = \exp(-9.066 + 0.097 * \text{SF} - 0.573 * \text{protected} - 0.977 * \text{speed40})(\text{totalADT})^{0.951} \qquad k = 0.582$$

where

SF = indicator for jurisdiction in California (1 = San Francisco; 0 otherwise) and
speed40 = indicator for maximum posted approach speed (1 = 40 mph or more; 0 otherwise).

In addition to treatment and reference sites, California and Maryland provided data on additional intersections that were signalized throughout the entire study period, but for which signal timing data were available for only a portion of the study period. These data were combined with the reference sites and data from the treatment sites in a cross-sectional analysis to investigate the individual yellow and all-red phases with respect to the ITE recommended practice.

The EB analysis was used to investigate the safety effects of modifications to the total change interval with respect to the ITE recommended practice. Because the sample size was relatively small, it was not possible to use the EB method to investigate the individual yellow and all-red phases with respect to the ITE recommended practice. A cross-sectional study was made instead, in which negative binomial regression models were used to analyze a larger sample of signalized intersections with various combinations of yellow and all-red phases. The cross-sectional analysis was based on a total of 916 site-years where the specific yellow and all-red times were known for each year.

Results for Increasing the Change Interval

The modifications to the yellow and all-red times were not equivalent for all sites. This lack of equivalence applies to both the existing conditions and the increase in the yellow or all-red intervals (or

TABLE 3 CMFs for Increasing Yellow and All-Red Intervals

Treatment	Crash Type	Severity	Expected Crashes Without Treatment	Observed Crashes With Treatment	CMF (SE of CMF)	Average Increase (min., max.) Yellow	Average Increase (min., max.) All-Red	Sites with No All-Red Before
Increase yellow and all-red (11 sites)	All	All	59.2	59	0.991 (0.146)	0.8 (0.5, 1.6)	1.2 (1.0, 2.0)	11
	All	Injury/fatality	52.69	54	1.020 (0.156)			
	Rear end	All	15.09	17	1.117 (0.288)			
	Angle	All	26.69	26	0.961 (0.217)			
Increase yellow only (five sites)	All	All	56.4	65	1.141 (0.177)	1.0 (1.0, 1.0)	na	1
	All	Injury/fatality	34.0	37	1.073 (0.216)			
	Rear end	All	22.1	21	0.934 (0.237)			
	Angle	All	19.7	22	1.076 (0.297)			
Increase all-red only (14 sites)	All	All	228.7	183	0.798 (0.074)*	na	1.1 (1.0, 2.0)	10
	All	Injury/fatality	96.8	84	0.863 (0.114)			
	Rear end	All	70.1	57	0.804 (0.135)			
	Angle	All	54.3	53	0.966 (0.164)			

NOTE: SE = standard error; min. = minimum; max. = maximum; na = not applicable.
*$p = .05$ (statistically significant at the .05 level).

both). For example, several of the intersections did not include an all-red phase in the before condition. For sites where both the yellow and all-red times were increased, the average increases in the yellow and all-red times were 0.8 s (minimum of 0.5 s and maximum of 1.6 s) and 1.2 s (minimum of 1.0 s and maximum of 2.0 s), respectively. For sites where only the yellow interval was increased, the increase in yellow time was 1.0 s at all sites. For sites where only the all-red interval was increased, the average increase in the all-red time was 1.1 s (minimum of 1.0 s and maximum of 2.0 s). For sites where the total change interval was increased but still smaller than the ITE recommended practice, the average increase was 0.9 s (minimum was 0 s and the maximum was 1.5 s). For sites where the total change interval was increased and did exceed the ITE recommended practice, the average increase was 1.6 s (minimum was 1.0 s and maximum was 3.0 s).

Based on the rigor of the EB method, and the general insignificant results of the cross-sectional analysis, the recommended CMFs were taken from the EB analysis. The EB before–after analyses indicated a significant reduction in total, injury, and rear-end crashes under various scenarios. Specifically, the EB analysis indicated

- A statistically significant reduction (at the .05 level) in total crashes as a result of increasing the all-red phase only,
- A statistically significant reduction (at the .05 level) in total and injury crashes as a result of increasing the total change interval to be smaller than the ITE recommended practice,
- A statistically significant reduction (at the .05 level) in rear-end crashes as a result of increasing the total change interval to be greater than the ITE recommended practice, and
- Statistically insignificant changes (at the .05 level) in angle crashes under all scenarios investigated.

Table 3 shows the CMFs and standard errors for total, injury, rear-end, and angle crashes as they relate to increasing the yellow or all-red intervals (or both). Table 4 shows similar results for increasing the total change interval. Each table also indicates the average increase in the respective interval, the applicable range of values, and the number of sites without an all-red phase in the before period. The number of sites in this evaluation was limited, and, therefore, the results should be treated with due caution.

As noted earlier, the results of the EB analysis were supplemented with a cross-sectional analysis. This supplementation was necessary because several sites did not have changes to the yellow or all-red phases during the study period. Also, some of the sites with modifications to the change interval had limited after-period data. A secondary objective of this supplemental investigation was to examine

TABLE 4 CMFs for Increasing the Change Interval

Treatment	Crash Type	Severity	Expected Crashes Without Treatment	Observed Crashes With Treatment	CMF (SE of CMF)	Average Increase in Change Interval (min., max.)	Sites with No All-Red Before
Increase change interval (<ITE) (12 sites)	All	All	217.2	159	0.728 (0.077)*	0.9 (0, 1.5)	11
	All	Injury/fatality	98.9	66	0.662 (0.099)*		
	Rear end	All	80.2	69	0.848 (0.142)		
	Angle	All	32.7	28	0.840 (0.195)		
Increase change interval (>ITE) (15 sites)	All	All	177.3	164	0.922 (0.089)	1.6 (1.0, 3.0)	10
	All	Injury/fatality	102.0	96	0.937 (0.114)		
	Rear end	All	50.7	33	0.643 (0.130)*		
	Angle	All	69.6	75	1.068 (0.156)		

*$p < .05$ (statistically significant at the .05 level).

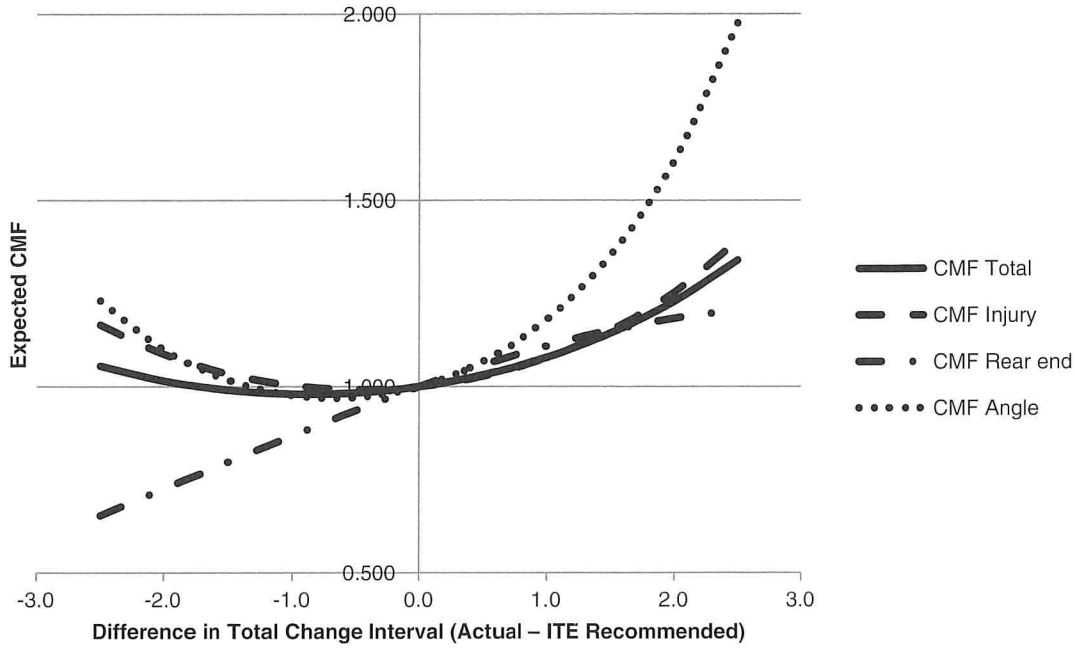


FIGURE 1 CMF trend with respect to difference in actual and ITE total change intervals.

the comparability of results of before–after and cross-sectional studies, a subject of topical interest in CMF development.

For the cross-sectional analysis, a negative binomial regression model was developed to estimate the safety effects of various change intervals at signalized intersections. The general functional form of the model assumed for this analysis is shown in Equation 1.

ADT and variables representing the change interval were included in every cross-sectional model developed for this evaluation. Once the models were estimated, the coefficient for the variable of interest was exponentiated to estimate the CMF. Additional variables were considered on the basis of available data and included in the models if the following conditions were met: first, the variable significantly improved the model; second, the effect of the variable was intuitive (e.g., crashes increase as the number of approaches increases).

The following additional variables were considered in the model development:

- Number of intersection approaches (three- or four-legged indicator),
- Signalized intersection phasing (permissive, protected, or protected-permissive indicators),
- Area type (all sites were urban in nature),
- Posted speed on major and minor approaches,
- Number of approach lanes,
- Width of approach lanes, and
- Presence and width of median on approach.

These variables entered the model form as adjustments to the base value of α in Equation 1. The base value of α was estimated for a particular baseline condition (e.g., three-legged, permissive phasing). When the condition of the intersection was anything other than baseline, an adjustment was applied to the base value of α. The parameter values (βs) indicate the magnitude and direction of the adjustment to the base α value.

The results from the two analyses are relatively consistent when the difference in the total change interval is compared. The results of the cross-sectional analysis show a U-shaped trend in the expected CMF, as shown in Figure 1. Specifically, there appears to be a safety benefit as the difference between the actual and ITE recommended change interval approaches zero. The results from the EB analysis similarly indicate a safety benefit as the total change interval is increased and approaches the ITE recommended practice and a less pronounced safety benefit as the total change interval is increased and exceeds the ITE recommended practice (i.e., a U-shaped trend). Although the general trends are consistent with the EB analysis, the coefficients are highly insignificant.

The key point of the analysis is that a constant CMF value may not accurately represent the safety impact of a treatment. Rather, it is conceivable that the safety impact could change over the range of values for a given treatment. If there is evidence of a significant nonlinear trend, as is the case here, the relationship is better defined by a series of CMFs or a crash modification function rather than a single CMF.

CONCLUSIONS AND RECOMMENDATIONS

There was a general safety benefit to the installation of DSWFs. The cross-sectional analysis showed a consistent reduction in total crashes and indications (considering limited sample sizes) that these reductions were in several crash types, including injury, rear-end, angle, and heavy-vehicle crashes. Supplemental before–after analyses generally corroborated the cross-sectional results, although there was a discrepancy that indicated an increase in rear-end crashes. With the exception of this crash type, the CMF results in Table 1 are ready for implementation when DSWF is being considered at signalized intersections with pertinent sight distance issues and for consideration in updating CMFs in the *Highway Safety Manual* (*12*).

For change interval modification, the before–after study results showed significant reductions (at the 5% level) in total, injury, and rear-end crashes under various scenarios. The results in Tables 3 and 4 are recommended for use in considering change interval modifications where there are related safety issues and for consideration in updating CMFs in the *Highway Safety Manual* (*12*). Specifically, these results indicate statistically significant reductions (at the .05 level) in

- Total crashes as a result of increasing the all-red phase only,
- Total and injury crashes as a result of increasing the total change interval to be smaller than the ITE recommended practice, and
- Rear-end crashes as a result of increasing the total change interval to be greater than the ITE recommended practice.

The results from a supplemental cross-sectional analysis were relatively consistent when the difference in the total change interval was compared. The results of the cross-sectional analysis show a U-shaped trend in the expected CMF, suggesting a safety benefit as the difference between the actual and ITE-recommended change intervals approaches zero. This variability should be captured by a series of CMFs, or a crash modification function, rather than by a single CMF; this issue should be addressed in future research.

ACKNOWLEDGMENTS

This paper is based on research conducted under NCHRP Project 17-35, Evaluation of Safety Strategies at Signalized Intersections. The authors express their gratitude to the members of the project panel for their guidance and patience during the performance of this research. This study could not have been completed without the data kindly provided by officials in the states of Nevada, North Carolina, and Virginia; the cities of San Diego and San Francisco; and the Maryland counties of Howard and Montgomery.

REFERENCES

1. Antonucci, N. D., K. K. Hardy, K. L. Slack, R. Pfefer, and T. R. Neuman. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan. Volume 12: A Guide for Reducing Collisions at Signalized Intersections.* Transportation Research Board of the National Academies, Washington, D.C., 2004.
2. Srinivasan, R., J. Baek, S. Smith, C. Sundstrom, D. Carter, C. Lyon, B. Persaud, F. Gross, K. Eccles, A. Hamidi, and N. Lefler. *NCHRP Report 705: Evaluation of Safety Strategies at Signalized Intersections.* Transportation Research Board of the National Academies, Washington, D.C., 2011.
3. ITE Technical Committee 4A-16. *Proposed Recommended Practice: Determining Vehicle Change Intervals.* ITE Publication No. RP-016. Institute of Transportation Engineers, Washington, D.C., 1985.
4. Retting, R. A., J. F. Chapline, and A. F. Williams. Changes in Crash Risk Following Re-Timing of Traffic Signal Change Intervals. *Accident Analysis and Prevention,* Vol. 34, No. 2, 2002, pp. 215–220.
5. Sayed, T., H. Vahidi, and F. Rodriquez. Advance Warning Flashers: Do They Improve Safety? In *Transportation Research Record: Journal of the Transportation Research Board, No. 1692,* TRB, National Research Council, Washington, D.C., 1999, pp. 30–38.
6. Radalj, T. Traffic Safety Effects of Advanced Warning Flashing Lights at Intersections on High-Speed Roads, Perth Metropolitan Area. *Proc. Road Safety Research, Policing, and Education Conference,* Sydney, Australia, 2003. http://www.rsconference.com/pdf/RS030129.pdf.
7. McCoy, P. T., and G. Pesti. Improving Dilemma-Zone Protection of Advance Detection with Advance-Warning Flashers. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1844,* Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 11–17.
8. Simpson, C. L. *Evaluation of Advance Activated Warning Signs.* North Carolina Department of Transportation, Transportation Mobility & Safety Division, Raleigh, 2010.
9. Gross, F., B. Persaud, and C. Lyon. *A Guide to Developing Quality Crash Modification Factors.* Publication FHWA-SA-10-032. FHWA, U.S. Department of Transportation, 2010.
10. Hauer, E. *Observational Before–After Studies in Road Safety: Estimating the Effect of Highway and Traffic Engineering Measures on Road Safety.* Pergamon Press, Oxford, U.K., 1997.
11. Hauer, E. Cause, Effect and Regression in Road Safety: A Case Study. *Accident Analysis & Prevention,* Vol. 42, No. 4, 2010, pp. 1128–1135.
12. *Highway Safety Manual.* AASHTO, Washington, D.C., 2010.

The Traffic Control Devices Committee peer-reviewed this paper.

Evaluation of Comprehension and Legibility of International and Domestic Nonstandard Symbol Signs

Bryan J. Katz, Erin E. Dagnall, and Cara C. O'Donnell

This paper documents a research project that evaluated proposed new traffic signs to ensure that the signs were effective when human factors requirements were taken into consideration. The following sign messages were selected for symbol development and evaluation: combination horizontal alignment–advisory speed, congestion ahead, cross-street preferential lane warning, do not enter, do not pass, electric-vehicle charging station, fallen rocks, flagger ahead, low-shoulder warning, maximum width, no left turn ahead, railroad crossing on leg of roundabout, road narrows, survey crew, trolley crossing, uneven lanes, and winery. The team conducted 96 surveys to determine driver comprehension and legibility for each alternative sign design. The surveys were administered to the general driving public at the Turner–Fairbank Highway Research Center in McLean, Virginia. On the basis of driver input and the results of the comprehension and legibility testing, the research team provided recommendations on symbols that should be included in the next edition of the *Manual on Uniform Traffic Control Devices.*

Traffic signs are important communication tools used to convey regulatory, warning, and guidance information to road users. The process of understanding user requirements for a new sign is particularly important in the development of symbol signs, which rely on a common nonverbal interpretation by a large and diverse population of drivers.

According to the *Manual on Uniform Traffic Control Devices* (MUTCD), "new symbol designs are adopted by the Federal Highway Administration based on research evaluations to determine road user comprehension, sign conspicuity, and sign legibility" (*1*). If any of these three major driver-related properties are inadequate, then the traffic control device will not provide adequate time for a proper response. Without proper response time, drivers will not be able to perceive problems and react to them in an adequate amount of time to maneuver their vehicles, which may ultimately lead to crashes.

LITERATURE REVIEW

There are many studies on the effectiveness of traffic signs, including evaluations of comprehension, legibility, and driver response. However, a standard or generally accepted methodology for the evaluation of symbol signs has yet to be identified.

Science Applications International Corporation, 6300 Georgetown Pike, F-215, McLean, VA 22101. Corresponding author: B. J. Katz, bryan.j.katz@saic.com.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 46–60.
DOI: 10.3141/2298-06

Dewar and Ells recognized a need for assessment methods to evaluate signs and other traffic control devices (TCDs) because there is little to suggest which currently used methods provide the best information (*2*). They identified several factors that should be evaluated: meaning, attention value, legibility, processing time, learnability, and influence on driver behavior. In a later paper, Dewar described six criteria as being important in the evaluation and design of symbol signs: legibility distance, understandability, conspicuity, learnability, glance legibility, and reaction time (*3*).

Understandability, hereafter referred to as "comprehension," has historically been measured a number of different ways. Alicandri and Wochinger had participants write their interpretation of the sign meanings and indicate what action they would take if the signs were seen on the roadway (*4*). Katz et al. used a similar procedure, except that they asked multiple-choice questions following participants' initial interpretation of sign meanings (*5*). The multiple-choice test was used to examine whether participants made problematic inferences about different signs (e.g., whether an animal presence sign with a flashing beacon turned off meant that no deer were present). In both cases, images of the signs were used without a background or roadway scene.

Dewar and Ells indicated that before-and-after studies are one of the most frequently used methods for evaluating signs; however, they also pointed out that there are several problems with this method (*2*). They suggested that three possible methods of evaluating signs included a field study conducted under normal driving conditions, a modified field study that used scaled-down signs, and a laboratory experiment to determine reaction time. They compared the three techniques and determined that the overall trends and relationships were similar, although the actual distances obtained in the simulator were shorter than those observed in the field.

Desrosiers performed field and laboratory investigations to determine the effectiveness of traffic signs (*6*). The author stated that laboratory studies eliminate problems dealing with environmental variables (weather, light, and traffic conditions), reduce the time required to gather data, and provide researchers with additional control over the experiment. It was concluded that laboratory tests could replace field tests, but a correction factor would have to be applied to distances obtained in the laboratory to obtain the same legibility distances observed in the field.

Sign research for both comprehension as well as recognition distances has been performed at the Turner–Fairbank Highway Research Center (TFHRC) in the past, as shown in Philips et al. (*7*), Alicandri and Wochinger (*4*), and Mahach et al. (*8*). The study by Philips et al. dealt solely with the use of the Sign Simulator Laboratory for determining comprehension and recognition distances. It was determined that relative recognition distances could be found in the

simulator but actual recognition distances could not be obtained without further validation.

For comprehension testing, previous researchers looked at both open-ended and multiple-choice responses to obtain information about various sign alternatives. This study incorporates similar methods to determine whether or not participants understand the general meaning of the signs and specific characteristics of the signs. The research indicates that TFHRC's Sign Simulator Laboratory will provide acceptable data for relative recognition distances; however, scale factors may be required to provide actual recognition distances.

RESEARCH GOALS

The goals of this study were to

- Identify candidate text and symbol signs,
- Evaluate driver comprehension of candidate signs,
- Measure the legibility distance of candidate signs, and
- Provide recommendations on signs that merit consideration for addition to the MUTCD.

SIGN CATEGORIES

Examples of various domestic and international symbol signs were gathered and studied before the development of new symbol-sign designs for this study. Some sign categories already have a version that either exists in the MUTCD or has been proposed for addition to the MUTCD. The remaining alternatives examined here were selected after a literature review and survey.

The following paragraphs give a description and background on each sign category examined in this study. The sign alternatives evaluated are shown in Table 1. The intended meaning for each sign is described and was used by researchers to determine, on the basis of participant responses to comprehension testing, if the signs were understood.

Combination Horizontal Alignment–Advisory Speed Messages

The intended purpose of combination horizontal alignment–advisory speed signs is to confirm the beginning of a horizontal alignment change, or curve, in the roadway and to remind road users of the advisory speed for the curve. The current MUTCD includes a standard horizontal alignment sign (sign W1-1 or W1-2) and states that this sign should be located at the advanced placement distance. The alternatives tested in this study differ from the current standard signs because the advisory speed is included on the sign rather than on a plaque below the sign. There has been no research on comprehension of the numerals on the sign [without the term "mph" (miles per hour)]; therefore, participants must show an understanding of both the alignment change and advisory speed in order to fully comprehend this sign.

Congestion Ahead

The "Congestion Ahead" sign is used to show that there may be congestion in the roadway ahead. A "Be Prepared To Stop" text-based sign (W3-4) is included in the current MUTCD for this purpose. The MUTCD states that the sign may be placed downstream from a "Signal Ahead" sign to warn of stopped traffic caused by a traffic control signal or regularly occurring traffic congestion on a section of roadway. It may be supplemented with a warning beacon with a "When Flashing" plaque. There have been no proposed symbol signs for this message in the MUTCD, so a symbol was included for evaluation to determine whether it is suitable for addition to the MUTCD.

Do Not Enter

The "Do Not Enter" sign is to be used where traffic is prohibited from entering a roadway. The symbolic sign for this message first entered the MUTCD in 1971, replacing the former sign that contained a text-based message only. The symbol sign also includes the text within the sign. The issue researched here was whether the symbol alone is understood by road users.

Do Not Pass

The "Do Not Pass" sign is used to indicate the restriction that no passing is allowed on a particular roadway or stretch of roadway. The MUTCD currently has a text-based sign (R4-1), which may be used in addition to pavement markings. A symbol sign alternative was also examined in this study.

Electric-Vehicle Charging Station

The intended purpose of the electric-vehicle charging station sign is to inform drivers of a nearby charging station for their electric vehicles. The 2003 MUTCD had a symbol sign showing a gas pump with the letters "EV" superimposed on it (D9-11b), and the 2009 MUTCD has both the symbol sign and an optional educational word message plaque (D9-11bP). As there has been no research on the comprehension of the existing sign, this symbol, as well as other symbols proposed by various agencies, was included in this study.

Fallen Rocks

The "Fallen Rocks" sign is used to indicate that there may be fallen rocks in the roadway ahead. It is to be used in advance of an area that is adjacent to a hillside, mountain, or cliff where rocks frequently fall onto the roadway. The MUTCD adopted a text sign (W8-14) until such time that symbol alternatives are researched. The most common signs that states use to warn drivers of falling (or already fallen) rocks are text signs; therefore, this study evaluated recommended symbol signs as well.

Flagger Ahead

The "Flagger Ahead" sign is used to indicate that there is a flagger ahead. It is placed in advance of any point where a flagger is stationed to control road users. The MUTCD includes a sign legend with "Flagger" text, which may also include a distance legend on a supplemental plaque. The 2009 MUTCD includes flagger symbols (W20-7 and W20-7a) as an alternative to the text-based legend sign. These three alternatives were evaluated in this study.

TABLE 1 Tested Symbol and Sign Alternatives

Sign Category	Alternative 1	2	3	4	5
Combination horizontal alignment–advisory speed	36 × 36 in.	36 × 36 in.	36 × 36 in.	36 × 36 in.	36 × 36 in.
Congestion ahead	36 × 36 in.	36 × 36 in.	36 × 36 in.	None	None
Cross-street preferential lane warning	36 × 36 in.	36 × 36 in.	None	None	None
Do not enter	24 × 24 in.	24 × 24 in.	None	None	None
Do not pass	24 × 24 in.	36 × 48 in.	None	None	None
Electric-vehicle charging station	24 × 24 in.	24 × 24 in.	24 × 24 in.	24 × 24 in.	None
Fallen rocks	36 × 36 in.	36 × 36 in.	36 × 36 in.	None	None
Flagger ahead	36 × 36 in.	36 × 36 in.	36 × 36 in.	None	None
Low-shoulder warning	36 × 36 in.	36 × 36 in.	36 × 36 in.	36 × 36 in.	None

(continued)

TABLE 1 *(continued)* **Tested Symbol and Sign Alternatives**

Sign Category	Alternative 1	2	3	4	5
Maximum width	36 × 36 in.	36 × 36 in.	None	None	None
No left turn ahead	36 × 36 in.	None	None	None	None
Railroad crossing on leg of roundabout	36 × 36 in.	None	None	None	None
Road narrows	36 × 36 in.	36 × 36 in.	None	None	None
Survey crew	36 × 36 in.	None	None	None	None
Trolley crossing	36 × 36 in.	36 × 36 in.	None	None	None
Uneven lanes	36 × 36 in.	36 × 36 in.	None	None	None
Winery	24 × 24 in.	24 × 24 in.	None	None	None

Low-Shoulder Warning

Low-shoulder warning signs indicate a change in elevation between the travel lanes and the shoulder. The signs should warn drivers to take extra caution to not drive off the roadway onto the shoulder. The 2009 MUTCD includes the W8-9 "Low Shoulder" word message sign and the W8-17 symbolic shoulder drop-off sign with a W8-17P word message educational plaque. The manual states that the W8-9 sign should be used where the elevation difference between the travel lane and the shoulder is less than 3 in. and the W8-17 symbolic shoulder drop-off sign should be used where the elevation difference exceeds 3 in. This study examined whether drivers understood the symbolic sign and whether they understood the difference in severity of elevation difference that is intended to be communicated by the two different signs.

Maximum Width

The purpose of maximum width signs is to warn drivers that the roadway ahead has a maximum width and that vehicles exceeding that dimension will not fit the roadway. Currently there are no proposed or adopted signs for this message in the 2009 MUTCD.

No Left Turn Ahead

The purpose of signs indicating no left turn ahead is to give drivers an advance warning that the left-turn movement is prohibited at the upcoming intersection. The MUTCD has neither proposed nor adopted a sign for this message.

Railroad Crossing on Leg of Roundabout

The intended purpose of the railroad crossing on leg of roundabout sign is to inform road users that there is a railroad crossing on one of the legs of the upcoming roundabout. Drivers should exercise caution if they are traveling in that direction. Currently there are no proposed or adopted signs for this message in the MUTCD.

Cross-Street Preferential Lane Warning

The intended purpose of cross-street preferential lane warning signs is to give an advance warning that the upcoming cross street has a high-occupancy vehicle (HOV) lane or some other restricted lane and that general purpose right-turning traffic into that cross street must not turn into the restricted lane. Currently there are no proposed or adopted signs for this message in the MUTCD.

Road Narrows

The "Road Narrows" sign is used to indicate that the road narrows ahead. It is to be used in advance of a transition on two-lane roads where the pavement width is reduced abruptly to a width such that vehicles traveling in opposite directions must reduce speed to travel through the narrow portion of the roadway. The MUTCD contained symbolic versions of this sign before 2003, but they were removed because road users did not comprehend them. It has been recommended that a uniform sign design should be selected and used in advance of the condition and along the road sections with the narrowed features and that a maximum width sign should be used to warn of the actual width (*9*).

Survey Crew

The sign for a survey crew is used to indicate that there is a survey crew working in or adjacent to the roadway ahead. The MUTCD includes a "Survey Crew" word legend sign (W21-6); however, no known symbol has been proposed or adopted in the MUTCD for this sign.

Trolley Crossing Advance Warning

The purpose of the trolley crossing advance warning sign is to warn drivers that there is a trolley crossing ahead. There are no proposed or adopted signs in the MUTCD.

Uneven Lanes

The "Uneven Lanes" sign is used to indicate that there are uneven lanes ahead. In other words, there is a difference in elevation between adjacent lanes that are open to travel. The MUTCD includes a text-based sign (W8-11), but it has yet to recommend a symbol sign pending further research.

Winery

The purpose of the winery sign is to indicate the location of a winery. This sign is to be used on guide signs to direct drivers to a winery. The MUTCD has neither proposed nor adopted any sort of sign for this purpose; however, there are several states that use various symbols for this indication.

METHOD

Research Design

The research was conducted at TFHRC in the highway sign simulator. Software was used in conjunction with a projector to display the images of highway signs onto a frosted glass screen. Signs were evaluated for comprehension and legibility. The software used for legibility distance testing was designed to gradually increase the size of the sign, emulating how that sign would appear when a driver was starting from a specified distance and driving toward the sign. Sign size, driving speed, and start distance were manipulated to make the representation as accurate as possible. The size of the sign when it became legible could then be translated into distance. Both the software for comprehension and for legibility electronically collected the required data and saved output data files for analysis.

Comprehension

The comprehension portion was a three-stage process in which participants provided open-ended responses, multiple-choice responses, and rankings of the signs. For the open-ended and multiple-choice sections, participants were shown only one sign alternative within each sign category. For each section, participants were shown one

sign; the sign remained on the screen for as long as they needed to provide their response. In the ranking section, participants were shown all alternatives for a given sign category and asked to rank each sign on how well they thought it showed the intended meaning of the sign. Participants completed all three comprehension sections for a given sign category before moving on to the next sign category.

Open-Ended Responses When the sign appeared on the screen the participants were asked, "What does this sign mean to you?" They were to respond aloud with what they thought the sign was trying to convey. A researcher transcribed their responses.

This process was followed by additional questions such as "What action should you take?" "How would this sign change your behavior?" and "Where would you expect to see a sign like this?" depending on the type of sign and which question(s) were most applicable. The goal of these questions was to further clarify the participants' understanding based on their response to the first question, or to inquire about a different aspect of the sign.

Multiple-Choice Responses Upon completion of the open-ended questions, participants were provided with three to four definitions of the sign (only one of which was correct) and asked to select the option that best described the meaning of the sign.

Ranking Finally, participants were shown all sign alternatives for the current sign category and were told the intended meaning of the sign. Each alternative was then presented on the screen individually, with a scale ranging from 1 to 7 in which 1 represented "would not work at all," 4 represented "might work," and 7 represented "would work very well." Participants were instructed to rank each sign on how well they thought it would work to show the intended meaning of the sign. They were instructed to rank each sign individually. In other words, two signs could have the same ranking; the participants were not to order the signs from best to worst and vice versa.

Legibility

Legibility distance was considered the maximum distance at which the participant could read text, decipher the elements of the sign, or both. For the legibility distance evaluation, participants were presented every sign option of each sign category, for a total of 41 test signs. There were 12 distracter signs added to counteract the guessing that could occur if a participant remembered some of the comprehension testing. The distracter signs included stop, yield, intersection, hospital, and airport signs, as well as signs that read "Railroad Crossing," "Fire Station," "Slower Traffic Keep Right," "No U-turn," "Deer Crossing," "Road Work," and "Dead End." The 53 signs were randomized, with the exception of the stop, yield, and railroad signs, which were always presented first as practice signs.

The signs were shown one at a time. Except for the sign itself, the entire projection screen was black. Each sign presentation began at a simulated distance of 1,000 ft (304.8 m). The sign then expanded in size to simulate an approach speed of 35 mph. Participants were instructed to keep their eyes on the sign and to push a button on the table in front of them when they could identify the sign. The instructions emphasized that this was not a comprehension test. Participants were asked to push the button immediately when they could make out what was on the sign, after which they would have time to describe the sign. When the button was pressed, the sign was stopped and the projection screen went completely black.

After the participant described the sign, the researcher determined if his or her response was correct. Correctness was deemed to be anything that confirmed that the sign was legible to the participant. If the response was correct, a new trial was begun with a different sign. If the response was incorrect, the same sign reappeared where it was stopped; it continued to increase in size until the button was pressed again and the sign was legible. If the sign size reached the full screen without the participant making a correct response, the trial was terminated. At that point the next trial would begin.

Participants

Participants were recruited from the Washington, D.C., metropolitan area. Of the 96 participants, half were 19 to 56 years of age (mean = 37 years), and half were 57 years or older. Each age group consisted of equal numbers of males and females. All participants possessed a valid driver's license and passed a vision screening with a minimum 20/40 acuity in at least one eye (corrected if necessary). Participants were paid $45 upon completion of the study.

RESULTS, FINDINGS, AND RECOMMENDATIONS

This section summarizes the results and findings of this study and explains the recommended alternatives that are effective when driver comprehension and legibility requirements are taken into consideration. All tests were evaluated with maximum probability of Type I error of 0.05. The alternatives that are referenced throughout this section can be found in Table 1.

Comprehension

Statistical testing for comprehension was based on the open-ended responses only, with multiple-choice and ranking percentages used as supplemental information.

Open-Ended Responses

Open-ended responses of the participants were evaluated on whether or not they reflected a correct understanding of the sign. The sign categories, alternatives in each category, and their respective mean comprehension scores based on open-ended responses are shown in Table 2.

Multiple-Choice Responses

The results of the multiple-choice section are shown in Table 2.

Rankings

The ranking data were used as supplemental subjective information and thus are not discussed in detail in the following individual sign summaries. Table 2 shows the mean rankings.

Legibility

The legibility distance results are shown in Figure 1.

TABLE 2 Comprehension Results

Sign Category	Comprehension Category	Comprehension Results (%), by Alternative				
		1	2	3	4	5
Combination horizontal alignment–advisory speed	Open-ended response: understood	56.3	100	100	81.3	100
Congestion ahead	Open-ended response: understood	100	100	78.10	—	—
	Multiple choice response					
	Parked cars on side of road	0	0	3.1	—	—
	Passing zone	0	0	3.1	—	—
	Congestion ahead[a]	100	75	93.8	—	—
	Vehicle checkpoint ahead	0	25	0	—	—
	Average ranking[b]	6.68	5.59	3.45	—	—
Cross-street preferential lane warning	Open-ended response					
	Understood general restriction (bike, bus, HOV)	75	62.5	—	—	—
	Understood HOV restriction	56.3	29.2	—	—	—
	Multiple choice response					
	Right turns limited by HOV lanes[a]	66.7	52.1	—	—	—
	Bicycle lane only in specified lane	8.3	25	—	—	—
	Right turns allowed	22.9	22.9	—	—	—
	HOV open in all lanes	2.1	0	—	—	—
	Average ranking[b]	4.26	4.04	—	—	—
Do not enter	Open-ended response: understood	68.8	100	—	—	—
	Multiple choice response					
	Yield	8.3	0	—	—	—
	Do not enter[a]	83.3	100	—	—	—
	Stop	8.3	0	—	—	—
	Enter	0	0	—	—	—
	Average ranking[b]	3.79	6.97	—	—	—
Do not pass	Open-ended response: understood	47.9	97.9	—	—	—
	Multiple choice response					
	Caution when passing	6.3	0	—	—	—
	Do not pass[a]	62.5	100	—	—	—
	No side-to-side parking	16.7	0	—	—	—
	Only one lane ahead	14.6	0	—	—	—
	Average ranking[b]	3.34	6.84	—	—	—
Electric-vehicle charging station	Open-ended response: understood	16.7	70.8	25	8.3	—
	Multiple choice response					
	Electronic device charging station	0	8.3	4.2	0	—
	Gas station	25	4.2	4.2	0	—
	Tire pressure station	4.2	0	0	0	—
	Electric-vehicle charging station[a]	70.8	87.5	91.7	100	—
	Average ranking[b]	4.05	6.05	3.75	3.54	—
Fallen rocks	Open-ended response: understood	100	100	96.9	—	—
	Multiple choice response					
	Watch for fallen rocks[a]	100	100	100	—	—
	Watch for broken pieces off building	0	0	0	—	—
	Rock-climbing area ahead	0	0	0	—	—
	Cliff-jumping area ahead	0	0	0	—	—
	Average ranking[b]	5.93	6.18	4.90	—	—
Flagger ahead	Open-ended response: understood	96.9	93.8	100	—	—
	Multiple choice response					
	School zone ahead	0	0	0	—	—
	Flagger ahead[a]	78.1	90.6	93.8	—	—
	Crossing guard ahead	21.9	9.4	6.3	—	—
	Information center ahead	0	0	0	—	—
	Average ranking[b]	6.01	5.94	6.08	—	—
Low-shoulder warning	Open-ended response: understood	83.3	91.7	4.2	16.7	—
	Multiple choice response					
	There is a change in elevation between two travel lanes	4.2	4.2	79.2	41.7	—
	There is a change in elevation between travel lanes and shoulder[a]	95.8	95.8	20.8	58.3	—
	Watch for metal plates in the travel lanes	0	0	0	0	—
	Average ranking[b]	5.51	5.60	4.65	4.05	—

(continued)

TABLE 2 *(continued)* **Comprehension Results**

Sign Category	Comprehension Category	Comprehension Results (%), by Alternative				
		1	2	3	4	5
Maximum width	Open-ended response: understood	87.5	95.8	—	—	—
	Multiple choice response					
	Minimum roadway width	8.3	2.1	—	—	—
	Tunnel clearance	12.5	10.4	—	—	—
	Maximum roadway width[a]	79.2	87.5	—	—	—
	Parking clearance	0	0	—	—	—
	Average ranking[b]	4.51	6.60	—	—	—
No left turn ahead	Open-ended response: understood	40.6	—	—	—	—
	Multiple choice response					
	No left turn ahead[a]	81.3	—	—	—	—
	No left turn	18.8	—	—	—	—
	No right turn ahead	0	—	—	—	—
	No right turn	0	—	—	—	—
	Average ranking[b]	5.85	—	—	—	—
Railroad crossing on leg of roundabout	Open-ended response: understood	90.6	—	—	—	—
	Multiple choice response					
	Train circle ahead	2.1	—	—	—	—
	Roundabout crossing	7.3	—	—	—	—
	Railroad crossing at right leg of roundabout[a]	90.6	—	—	—	—
	Pedestrian crossing at right leg of roundabout	0	—	—	—	—
	Average ranking[b]	5.85	—	—	—	—
Road narrows	Open-ended response: understood	35.4	85.4	—	—	—
	Multiple choice response					
	Merging lanes	12.5	4.2	—	—	—
	Road narrows[a]	58.3	91.7	—	—	—
	Roadway merges to one lane	20.8	4.2	—	—	—
	Median ends	8.3	0	—	—	—
	Average ranking[b]	4.83	6.61	—	—	—
Survey crew	Open-ended response: understood	92.7	—	—	—	—
	Multiple choice response					
	Research park	0	—	—	—	—
	Scenic overlook	8.3	—	—	—	—
	Survey crew ahead[a]	91.7	—	—	—	—
	Film studio	0	—	—	—	—
	Average ranking[b]	6.01	—	—	—	—
Trolley crossing	Open-ended response: understood	97.9	95.8	—	—	—
	Multiple choice response					
	School bus crossing	2.1	0	—	—	—
	Train station	2.1	4.2	—	—	—
	Trolley crossing[a]	93.8	95.8	—	—	—
	Bus station	2.1	0	—	—	—
	Average ranking[b]	4.61	6.31	—	—	—
Uneven lanes	Open-ended response: understood	66.7	85.4	—	—	—
	Multiple choice responses					
	Uneven lanes[a]	89.6	97.9	—	—	—
	Soft shoulder	0	0	—	—	—
	No passing	4.2	2.1	—	—	—
	Road under construction	6.3	0	—	—	—
	Average ranking[b]	5.99	5.07	—	—	—
Winery	Open-ended response: understood	97.9	56.3	—	—	—
	Multiple choice response					
	Winery[a]	100	79.2	—	—	—
	Orchard	0	6.3	—	—	—
	Fruit stand	0	10.4	—	—	—
	Harvest center	0	4.2	—	—	—
	Average ranking[b]	6.09	3.75	—	—	—

NOTE: — = data not applicable.
[a]Correct response.
[b]Ranking on scale of 1 to 7, where 1 = would not work at all and 7 = would work very well.

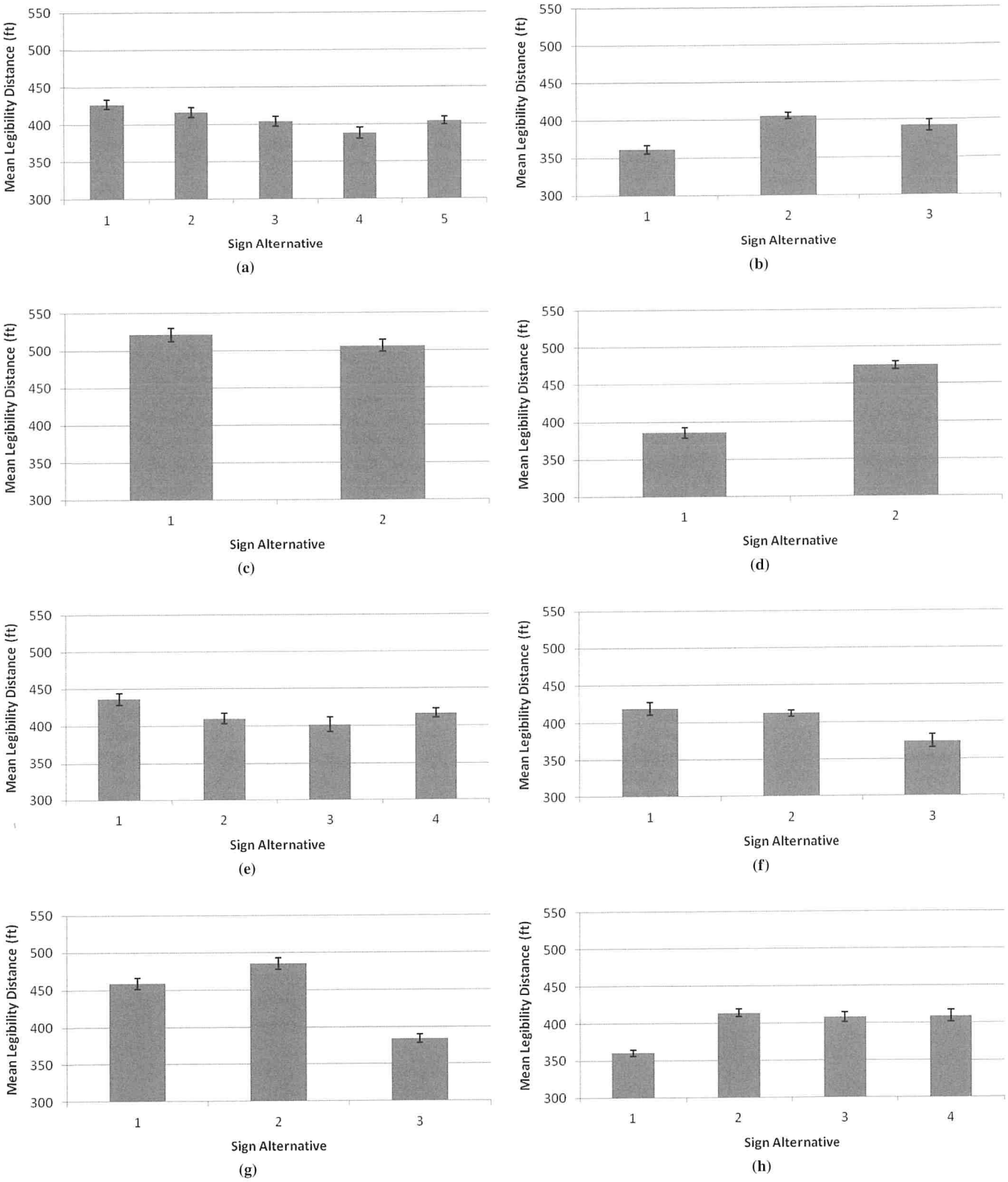


FIGURE 1 Legibility distance results: (*a*) combination horizontal alignment–advisory speed, (*b*) congestion ahead, (*c*) do not enter, (*d*) do not pass, (*e*) electric-vehicle charging station, (*f*) fallen rocks, (*g*) flagger ahead, and (*h*) low-shoulder warning.

(continued)

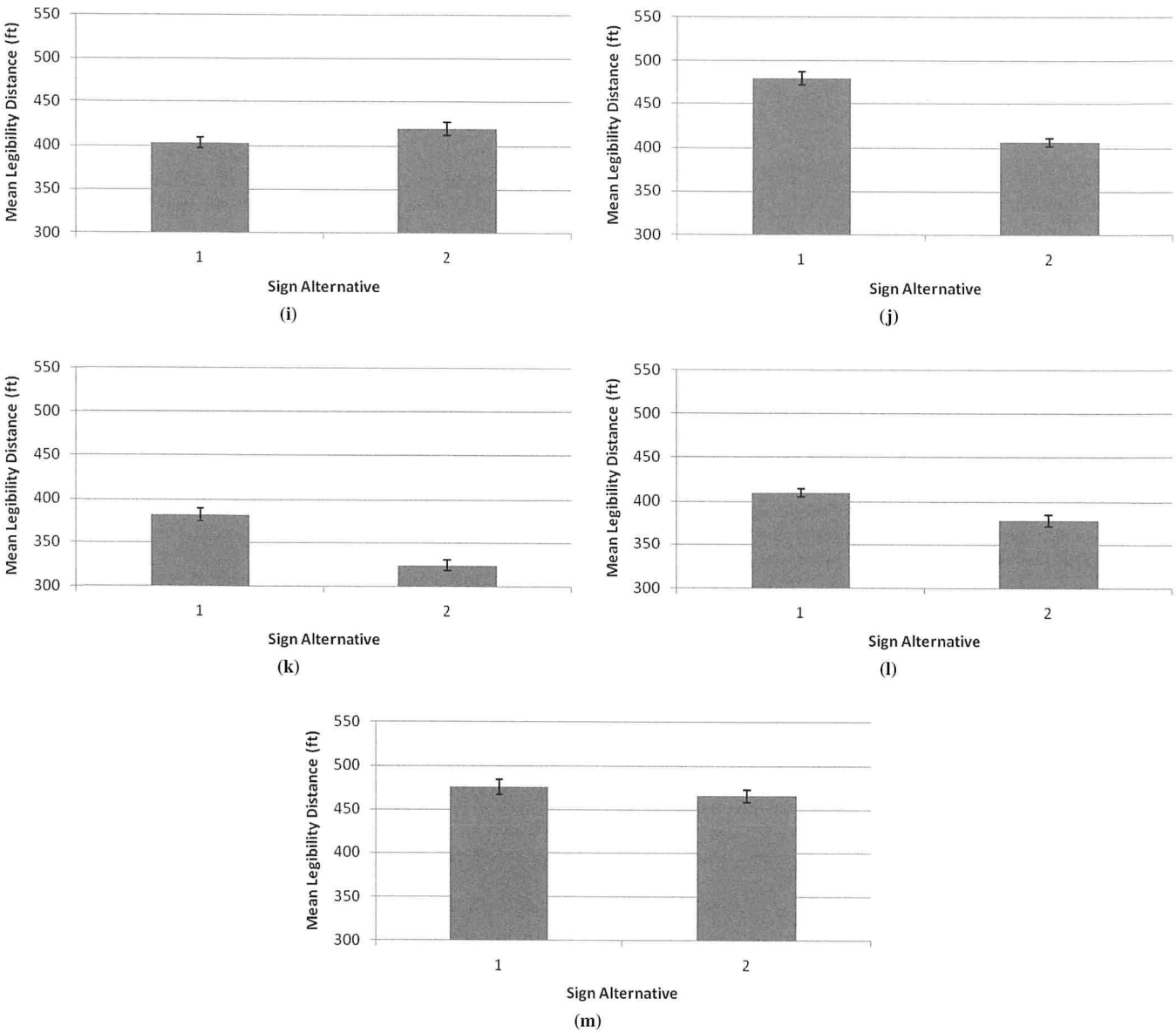


FIGURE 1 *(continued)* Legibility distance results: (*i*) maximum width, (*j*) road narrows, (*k*) trolley crossing, (*l*) uneven lanes, and (*m*) winery.

Sign Summaries and Recommendations

Combination Horizontal Alignment–Advisory Speed Messages

For combination horizontal alignment–advisory speed messages, comprehension varied significantly by sign alternative ($p < .001$); Alternative 1 was significantly less well comprehended than Alternatives 2, 3, and 5 ($Z = 2.57$, $p = .005$).

For Alternative 1, all incorrect responses by participants were the result of interpretation of the number as a representation of distance to the alignment change. With Alternative 4, 66.7% of incorrect responses occurred because participants interpreted the number as the degree, or sharpness, of the curve, and 33.3% of the participants thought it meant the driver must go 35 mph around the truck. However, of the 81.3% of participants who correctly understood Alternative 4, 30.8% thought the advisory speed applied to trucks only. The alignment sign alternatives each depict a different message, so the multiple-choice and ranking sections were not used to assess comprehension for this sign group.

The mean legibility distances were 427 ft (130.24 m), 417 ft (127.19 m), 404 ft (123.22 m), 389 ft (118.65 m), and 405 ft (123.53 m), respectively, for Alternatives 1, 2, 3, 4, and 5. Because each sign indicates a different message, an analysis to compare the five legibility distances would not necessarily be meaningful.

The selected alignment signs as tested appear to work well for providing roadway geometry information, but the corresponding numerals may not be appropriate where advisory speeds are less than 15 mph. Additional research may be needed to determine potential issues with numerals without the traditional term "mph." It is not known whether participants understood the difference between an advance warning and a warning at the start of a horizontal alignment

change. Because Alternative 4 resulted in a lower mean legibility distance, the sign should be enlarged in situations where it is used.

Congestion Ahead

For messages indicating congestion ahead, Fisher's exact test indicated that comprehension varied significantly by sign alternative ($p < .001$). The symbol version was not as well comprehended as the text-based alternatives.

A repeated-measures analysis of variance showed that the mean legibility distances differed significantly, with $F(1.62) = 32.24$, $p < .001$. The legend size in Alternative 2 at approximately 5 in. (13 cm) is larger than Alternative 1 at approximately 4 in. (10 cm), which makes the sign more legible from farther distances. The symbol used in Alternative 3 resulted in a lower mean legibility distance compared with Alternative 2. Alternative 1 had lower distances than both, presumably because the shorter letter height required fitting the word "congestion" on the sign.

Alternative 2 provides information about the appropriate action to take in some cases when there is congestion ahead (in other cases, the congested condition might require only a significant or abrupt reduction in speed but not a complete stop). Further, the sign has a broad purpose and can be used in other situations (such as vehicle checkpoints, toll plazas, or other locations where drivers need to stop). Therefore, if the intent is to make drivers aware of congestion specifically, the symbol in Alternative 3 may be combined with a text plaque to train drivers in the meaning. Then, the increase in legibility distance that is found in Alternative 3 compared with that of Alternative 1 will provide further benefit once drivers learn the meaning of the symbol.

Cross-Street Preferential Lane Warning

The open-ended responses for cross-street preferential lane warning signs were evaluated on two levels of comprehension. The first level, general restriction, was based on whether or not the participant understood that there was some type of restriction (HOV, bike lane, bus lane, etc.) The second level, HOV restriction, was based on whether or not the participant understood specifically the HOV restriction alone.

For general restriction, there was no significant difference in comprehension between the two alternatives ($\alpha = .05$ level). For HOV restriction, comprehension for Alternative 1 was significantly higher than that for Alternative 2 ($Z = 2.68$, $p = .007$). Because the alternatives indicate a different message, an analysis to compare the legibility distances was not practical.

Participants had a general understanding of a restriction, so the use of warning signs for preferential lanes at right-hand turns may be appropriate. However, because participants did not understand the form of restriction, another form of guide or regulatory sign showing lane configurations with arrows indicating "HOV only" and "Other Traffic" may be better alternatives.

Do Not Enter

For the "Do Not Enter" sign, the only difference between Alternatives 1 and 2 was the presence of the "Do Not Enter" legend in Alternative 2, which is similar to the current version in the MUTCD. The MUTCD does not use a black border around the sign, which was an oversight by the research team.

Comprehension for Alternative 2 was significantly higher than comprehension for Alternative 1 ($Z = 4.22$, $p < .001$). The mean legibility distance for Alternative 1 was significantly longer than that for Alternative 2, with $t(93) = 2.136$, $p = .035$.

Participants do not easily understand the meaning of the "Do Not Enter" sign without the text. Even with multiple-choice responses, some participants thought the meaning was to yield or to stop. The legibility distances were significantly different, but this difference is unlikely to be of practical significance, as both signs were legible from more than 500 ft (150 m). The existing sign (Alternative 2) should continue to be used; it is similar to the international version and therefore should be understood by those familiar with the international sign and by North American drivers. Additionally, the existing MUTCD standard without a black border should be used because the border on the alternatives tested should not result in any reduced level of comprehension or legibility.

Do Not Pass

For the "Do Not Pass" sign, comprehension was significantly higher for Alternative 2 than for Alternative 1 ($Z = 5.19$, $p < .001$). The mean legibility of the symbol sign (Alternative 1) was significantly less than that for the text sign, with $t(93) = 13.622$, $p < .001$.

The existing "Do Not Pass" sign, Alternative 2, should continue to be used until an improved symbol is developed. It may be possible to introduce an educational plaque to improve comprehension of the sign; however, detailed research should be performed to make sure that people understand the sign before the plaque is removed.

Electric-Vehicle Charging Station

A chi-square test of independence indicated that comprehension varied significantly by sign alternative, with $\chi^2(3, N = 96) = 26.63$, $p < .001$. Post hoc tests showed that Alternative 2 was comprehended better than the other three alternatives. Of all the incorrect responses given by participants for Alternative 1, 100% indicated an interpretation of the sign as a gas station sign.

The mean legibility distances of the alternatives differed significantly at $F(2.51) = 9.51$, $p < .001$. Alternative 1, which is currently in the MUTCD, performed the best, presumably because of its relatively simple design. The differences in mean legibility distance were not practically significant; however, Alternative 2 may be improved by widening the plug and separating it from the pump.

The researchers recommend that Alternative 2 be used for electric-vehicle charging stations and that the previously mentioned suggestions be made to improve its legibility.

Fallen Rocks

For this study, participants were correct whether they distinguished between "falling" and "fallen" rocks or not. There were no significant differences in comprehension of the three alternatives ($p > .05$).

The mean legibility distances of the sign alternatives differed significantly, with $F(2) = 21.04$, $p < .001$. There were no significant differences between Alternatives 1 and 2 at the $\alpha = .05$ level; however, both alternatives had better legibility than Alternative 3.

Alternatives 1 and 2 were both well understood and are both equally legible, so the symbolized message of Alternative 1 is preferred because of the increasing proportion of non-English speaking drivers.

Flagger Ahead

For the "Flagger Ahead" sign, Fisher's exact test indicated no significant differences in comprehension between sign alternatives, with $p = .771$. The mean legibility distances of the sign alternatives differed significantly, with $F(2) = 122.61$, $p < .001$. Both of the symbol signs (Alternatives 1 and 2) performed better than the text-based sign (Alternative 3), and the current standard sign (Alternative 2) performed better than the alternative symbol (Alternative 1).

All of these signs received high comprehension ratings; however, when participants were provided options, 22% of them thought that Alternative 1 might mean there was a crossing guard ahead, and for legibility distance, Alternative 2 performed the best. It is recommended that the current MUTCD standard (Alternative 2) be retained.

Low-Shoulder Warning

For low-shoulder warning signs, comprehension varied significantly by sign alternative, with χ^2 (3, $N = 96$) = 58.15, $p < .001$. Post hoc analyses showed that Alternatives 1 and 2 were significantly different from Alternatives 3 and 4. Participants did not understand the symbol alternatives. In many cases, participants thought that there was a change in elevation between two travel lanes.

Participants were also asked if they thought there was a difference between a shoulder drop-off and a low shoulder. Of the 55.2% participants who said there was a difference between them, the majority of the participants (84.9%) correctly attributed the difference to the level of severity, saying that a shoulder drop-off is more severe than a low shoulder.

The mean legibility distances of the alternatives differed significantly at $F(2.36) = 38.33$, $p < .001$. There were no significant differences between Alternatives 2, 3, and 4 at the $\alpha = .05$ level. Alternative 1 contains smaller text than Alternative 2 and was not legible from this same distance.

The use of the text-based alternatives is recommended. The sign reading "Shoulder Drop-Off" should be used where the difference in elevation between the travel lanes and shoulder is higher than that for "Low Shoulder."

Maximum Width

For maximum width signs, there was no significant difference in comprehension between the two alternatives at the $\alpha = .05$ level. Although the majority of participants responded correctly in understanding that this sign shows maximum width, there was disparity with regard to what this width was referring to. Responses included the road, lane, upcoming tunnel or bridge, or vehicle.

The mean legibility distance for Alternative 2 was significantly greater than that for alternative 1, $t(93) = 2.765$, $p = .007$, but the numerals are larger for this sign, so the difference does not have any practical significance.

On the basis of the comprehension results, either alternative would be acceptable to indicate maximum width; however, because the mean ranking was higher for Alternative 2, Alternative 2 is recommended.

No Left Turn Ahead

The sign concerning no left turn ahead was understood by 40.6% of participants. Those who did not understand the meaning of the sign tended to interpret it as meaning "No left turn; go straight ahead only." The mean legibility distance for the no-left-turn-ahead sign was 410 ft (125.05 m) with a standard error of 5.98.

On the basis of the comprehension results, use of the proposed sign without further evaluation or symbol refinement is not recommended. In the meantime, if a specific message is required, a word legend sign indicating "No Left Turn at Elm Street" or similar message should be used.

Railroad Crossing on Leg of Roundabout

The meaning of the sign concerning a railroad crossing on one leg of a roundabout was understood by 90.6% of participants. The mean legibility distance for the sign was 423 ft (129.02 m) with a standard error of 7.47.

Because the sign had both high comprehension and legibility, it is recommended as being appropriate for use. However, the symbol used to depict the roundabout is different from the standard roundabout warning sign, so practitioners will need to consider the effects of this difference before implementation.

Road Narrows

Comprehension of Alternative 2 of the "Road Narrows" sign was significantly higher than comprehension of Alternative 1 ($Z = 5.01$, $p < .001$). The mean legibility distance of Alternative 1 was significantly higher than that for Alternative 2, with $t(93) = 11.048$, $p < .001$.

Comprehension tests suggested that many drivers will not understand the symbol sign (Alternative 1); the legibility was much higher for Alternative 1. The text-based sign (Alternative 2) should be retained because it was more effective in communicating the message. A test of a modified version of the symbol that includes lane lines might help to determine whether that inclusion improves comprehension of the sign.

Survey Crew

The meaning of the sign concerning survey crews was understood by 92.7% of participants; the multiple-choice and rankings support this statement. The mean legibility distance for the survey crew symbol sign was 409 ft (124.75 m), with a standard error of 5.83. The symbol sign is recommended for use.

Trolley Crossing

For the signs concerning trolley crossings, there was no significant difference in comprehension between the alternatives at the

$\alpha = .05$ level. The mean legibility distance of Alternative 1 was significantly higher than that for Alternative 2, with $t(93) = 10.181$, $p < .001$. Based on these results, Alternative 1 is recommended for implementation.

Uneven Lanes

Comprehension of Alternative 2 was significantly higher than comprehension of Alternative 1 ($Z = 2.15$, $p = .032$) for the signs concerning uneven lanes. The mean legibility distance of Alternative 1 was significantly higher than that for Alternative 2, with $t(94) = 4.823$, $p < .001$.

Because the text "Uneven Lanes" is not well understood without a picture, use of Alternative 2 is recommended when a sign is needed, but the sign should be oversized to improve legibility distance.

Winery

For signs concerning a winery, comprehension of Alternative 1 was significantly higher than comprehension of Alternative 2 ($Z = 4.86$, $p < .001$). Multiple-choice and rankings supported this finding. The mean legibility distance of Alternative 2 was significantly higher than that for Alternative 1, with $t(94) = 48.96$, $p < .001$.

Because of the higher comprehension ratings of Alternative 1, it is recommended that a modified version of the sign be developed to increase legibility distance; however, the sign would be appropriate if enlarged or used only on low-speed roads.

CONCLUSIONS

In this study, the research team identified several alternative symbols for 17 traffic sign messages and evaluated the driver comprehension and legibility distance for many of the alternatives. The better-performing symbols signs are recommended for adoption in the MUTCD.

Table 3 shows the recommended alternative(s) for each of the symbols or signs evaluated in the study. In some cases, the research team is not recommending any of the symbols evaluated for implementation because of the poor comprehension levels found in the comprehension evaluation.

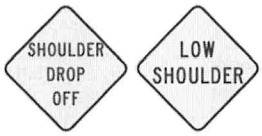
As stated previously, past research indicates that TFHRC's Sign Simulator Laboratory will provide acceptable data for relative recognition distances, although scale factors may be required to

TABLE 3 Recommended Symbol and Sign Alternatives

Sign Category	Recommended Alternatives
Combination horizontal alignment–advisory speed	Consider adding mph to sign to indicate advisory speed.
Congestion ahead	Use "Congestion Ahead" educational plaque with sign.
Cross-street preferential lane warning sign	No sign recommended
Do not enter	
Do not pass	
Electric-vehicle charging station	Improve design: widen cord and plug and separate cord away from pump to increase legibility.

(continued)

TABLE 3 *(continued)* **Recommended Symbol and Sign Alternatives**

Sign Category	Recommended Alternatives
Fallen rocks	
Flagger ahead	
Low-shoulder warning	 Choose sign on basis of elevation difference between shoulder and travel lanes.
Maximum width	
No left turn ahead	No sign recommended.
Railroad crossing on leg of roundabout	
Road narrows	
Survey crew	
Trolley crossing	
Uneven lanes	
Winery	Consider revisions to reduce complexity and increase legibility distance.

provide actual recognition distances. However, a field study would be required to effectively relate the Sign Simulator results to field results. For comparing alternatives, the Sign Simulator Laboratory is expected to provide the information required.

ACKNOWLEDGMENTS

The authors thank Amanda Emo of FHWA, and William Perez of Science Applications International Corporation (SAIC) for their management oversight on the project as well as Douglas Cobb and Mary Anne Bertola of SAIC for their technical input. This research was funded by the Traffic Control Devices (TCD) Pooled Fund Study (PFS), which focuses on systematic evaluation of novel TCDs and employs a consistent process that addresses human factors and operations issues for each TCD idea. By pooling resources and expertise instead of performing several independent research studies across the country, the PFS provides local and state agencies faster response to their needs and faster response to new technologies with the right assessment skills and tools to enable consistent TCD idea identification and evaluation. The PFS efforts address TCD issues identified by local and state jurisdictions, industry, and organizations and aid in compliance with the MUTCD rule-making process and incorporation of novel TCDs into the MUTCD.

REFERENCES

1. *Manual on Uniform Traffic Control Devices.* FHWA, U.S. Department of Transportation, 2009.
2. Dewar, R. E., and J. G. Ells. Comparison of Three Methods for Evaluating Traffic Signs. In *Transportation Research Record 503,* TRB, National Research Council, Washington, D.C., 1974, pp. 38–47.
3. Dewar, R. Criteria for the Design and Evaluation of Traffic Sign Symbols. In *Transportation Research Record 1160,* TRB, National Research Council, Washington, D.C., 1988, pp. 1–6.
4. Alicandri, E., and K. Wochinger. *Evaluation of Selected Potential MUTCD Signs.* Report FHWA-RD-00-053. FHWA, U.S. Department of Transportation, 2000.
5. Katz, B. J., G. K. Rousseau, and D. L. Warren. Comprehension of Warning and Regulatory Signs for Speed. Proc., Annual Meeting of the Institute of Transportation Engineers Seattle, Wash., 2003.
6. Desrosiers, R. D. Moving Picture Technique for Highway Signing Studies—An Investigation of its Applicability. *Public Roads,* Vol. 33, No. 7, 1965, pp. 143–147.
7. Philips, B., J. Fox, and R. Peters. Computer-Aided Optimization and Evaluation of Selected Signs. Presented at 77th Annual Meeting of the Transportation Research Board, Washington, D.C., 1998.
8. Mahach, K. R., K. Wochinger, R. Marshall, and D. Eppich. Sign Simulator Validated in FHWA Study. *Public Roads,* Vol. 63, No. 1, 1999, pp. 41–44.
9. Wainwright, W. S. *Synthesis of Non-MUTCD Traffic Signing.* Report FHWA-HRT-06-091. FHWA, U.S. Department of Transportation, 2005.

The Traffic Control Devices Committee peer-reviewed this paper.

Effect of Train Arrival Time on Crash Frequency at Highway–Railroad Grade Crossings

General Classification Regression Model

Shou-Ren Hu and Jhy-Pyng Lin

With the use of a general classification regression model, this study investigated the causal relationship between time to train arrival (TTA) and crash frequency at highway–railroad grade crossings. In particular, a stratified structure in the explanatory variables was used to avoid the collinearity problem generally confronted in linear regression models. TTA is a good estimate of rail sight distance and time to collision, and it could be used to predict crash frequency at a grade crossing. A 14-year crash data set accompanied by crossing inventory data including TTAs was collected for the empirical study. Study results indicated that a negative relationship between TTAs and crash frequencies was generally found for all types of trains. Similar causal relationships were also found in various combinations of both crossing attributes and crash characteristics. Sensitivity analysis on the variable combinations was also conducted to investigate the key risk factors that might result in traffic collisions at grade crossings. Policy implications based on the empirical study are discussed, and future research directions are recommended.

Highway–railroad grade crossings (HRGXs), which are spatial locations where highway and railway systems intersect with each other, can be dangerous if people do not take necessary actions and appropriate cautions near them. In 2002, HRGX and trespasser deaths accounted for 94.3% of all rail-related deaths in the United States (*1*). Similarly, grade-crossing users accounted for approximately one in seven accidental fatalities in the United Kingdom (*2*). In Taiwan, even though traffic collisions at HRGXs accounted only for approximately 10% of total rail-related incidents, crash severity in terms of the number of fatalities or injuries per traffic collision at HRGXs was 3.5 times higher than that of the entire Taiwanese rail system in 2006 (*3*). Figures 1 and 2 respectively show the crash frequency and casualties at HRGXs in Taiwan from 1997 through 2010 (*4*). As indicated in these two figures, although in the past few years the number of traffic collisions or severe casualties at HRGXs has been gradually decreasing, the safety records at HRGXs in Taiwan still have room for improvement. For instance, Figure 3 reveals that both the average HRGX collision rate in terms of the number of collisions per 100 crossings per year (CCY) and fatality rate in terms of the number of deaths per 100 crossings per year (DCY) are much higher than those in the compared countries. These higher rates indicate that the safety records of HRGXs in Taiwan are much lower than those in the international rail community and urgent action is needed to provide both rail and highway users with a safe traffic environment at grade crossings.

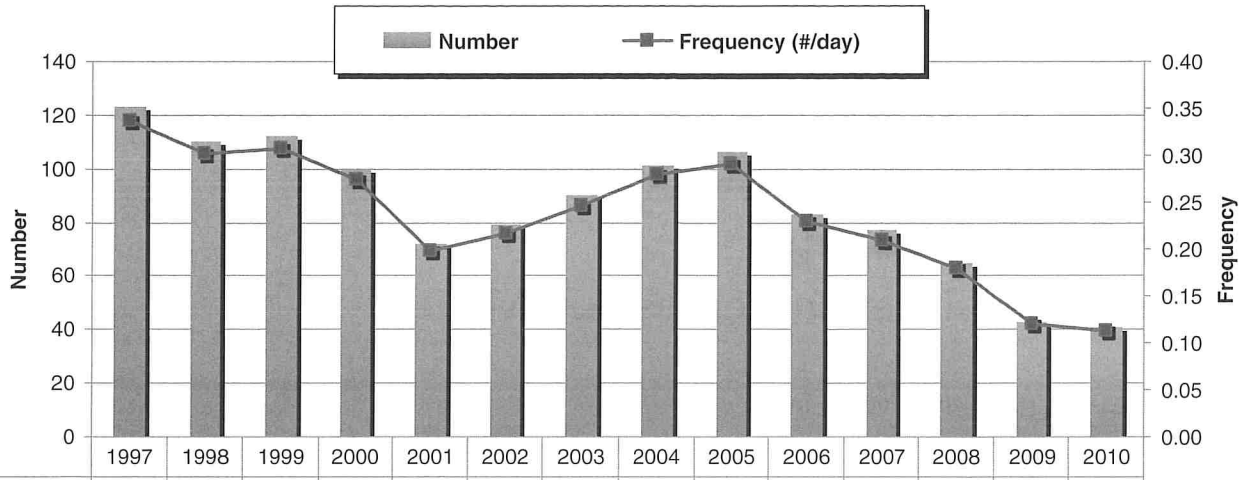
In addition to the severe impacts on different entities or individuals involved in an HRGX traffic collision, significant travel delays and social costs associated with HRGX crashes have been reported (*4*). Figure 4 illustrates the average train stay (defined as the average holding time of the collision train at the crash site) and train delay (defined as the total time of delay for the influenced trains that is due to a train collision incident) of traffic collisions at HRGXs from 2005 to 2010 in Taiwan. As shown in Figure 4, the train-stay time and train-delay time involved in a fatal train collision incident are longer than those of an injury- or damage-only incident. In addition, for a specific train collision incident, the total train-delay time of the influenced trains is much longer than that of the collision train, indicating that not only is there a cost associated with the individuals directly involved in the traffic collision, but there is also a significant social cost to the other rail passengers and highway users. These costs call for special attention in preventing traffic collisions at HRGXs.

Traffic collisions at grade crossings are mainly due to driving errors made by highway users. According to the grade-crossing crash records provided by the Taiwan Railways Administration (TRA) (*4*), among the causes for the traffic collisions at grade crossings in Taiwan, less than 1% of the overall cases are attributed to the malfunction or inappropriate operation of railway devices (e.g., gatekeeper errors, see Figure 5). In addition, near 90% of the traffic collisions at HRGXs are because of highway users' disregard for the HRGX warning devices. Among the potential risk factors that result in a traffic collision at HRGXs, sight distance is one of the main concerns that warrants further investigation. Berg et al. investigated the causal factors in HRGX accidents and found that the principal contributing factors were low driver expectancy of a hazard and inadequate quadrant sight distance (*5*).

The sight distance of a grade crossing either from the highway or railway side is thought to be one of the critical factors that determine the probability of a grade-crossing crash incident, especially

S.-R. Hu, Department of Transportation and Communication Management Science, National Cheng Kung University, No. 1 University Road, Tainan City 70101, Taiwan. J.-P. Lin, Department of Project Construction, Taiwan Railways Administration, Ministry of Transportation and Communications, Taiwan, 4F, No. 3 Beiping West Road, Taipei City 10041, Taiwan, and Department of Transportation and Communication Management Science, National Cheng Kung University, No. 1 University Road, Tainan City 70101 Taiwan. Corresponding author: S.-R. Hu, shouren@mail.ncku.edu.tw.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 61–69.
DOI: 10.3141/2298-07

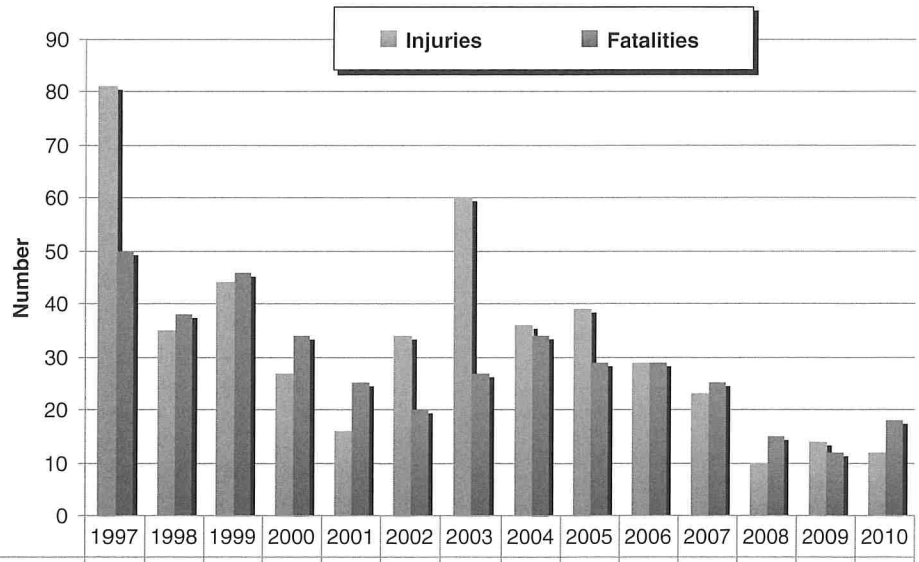


	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Number	123	110	112	100	72	79	90	101	106	83	77	65	43	41
Frequency (#/day)	0.34	0.30	0.31	0.27	0.20	0.22	0.25	0.28	0.29	0.23	0.21	0.18	0.12	0.11

FIGURE 1 Annual HRGX crash frequency in Taiwan.

for heavy vehicles. A Canadian study showed that approximately 9% to 17% of all grade-crossing crashes involved a heavy vehicle and many of these collisions resulted from visibility problems (*6*). FHWA's revised second edition of the *Railroad–Highway Grade Crossing Handbook* states that insufficient sight distance of an HRGX might result in a safety incident, and such an incident might possibly be caused by sight obstructions from buildings, vegetation, roadside commercial signage, utilities, and lighting poles (*7*). For this reason, the millennium edition of the *Manual on Uniform Traffic Control Devices* (MUTCD) recommended the use of warning devices (*8*). These devices may be passive elements, such as traffic signs (e.g., the crossbuck, inverted T, "Exempt," and "Do Not Stop on Track"), pavement markings, and rumble strips, or they may be active elements such as railroad signal gates, repeater signals, locomotive horns, and traffic signal interconnects. These devices are usually installed around a grade-crossing area to address the safety problem.

From the railway side, the braking time period to avoid a traffic collision is dependent on the train's speed and the distance from which the train starts braking. Traffic collisions at grade crossings are usually due to insufficient survival time periods of both the highway and railway sides. As a result of the mechanical characteristics of a locomotive, the braking distance is usually fixed when a locomotive engineer decides to make an emergency brake, and the maximum time to survive (TTS) for the highway user is correspondingly



	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Injuries	81	35	44	27	16	34	60	36	39	29	23	10	14	12
Fatalities	50	38	46	34	25	20	27	34	29	29	25	15	12	18

FIGURE 2 Annual HRGX crash casualties in Taiwan.

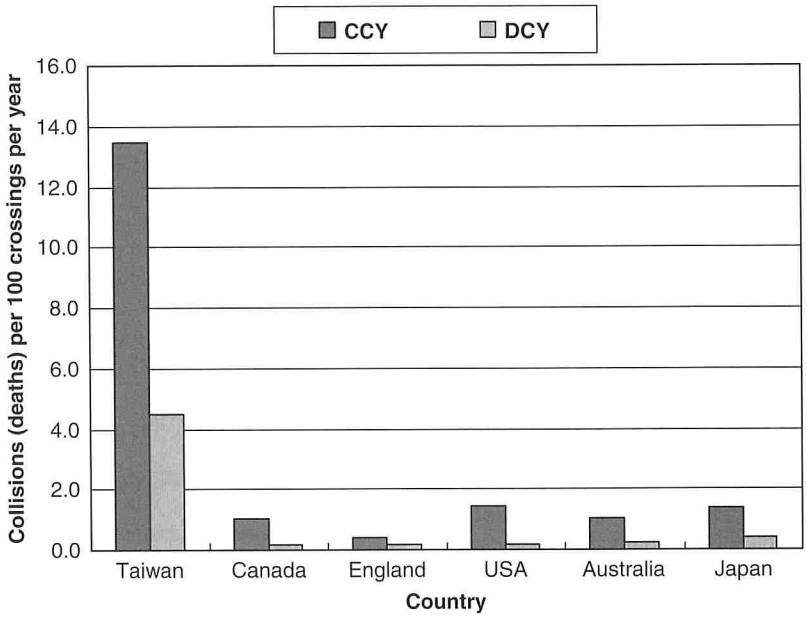


FIGURE 3 Comparisons of HRGX safety records of selected countries.

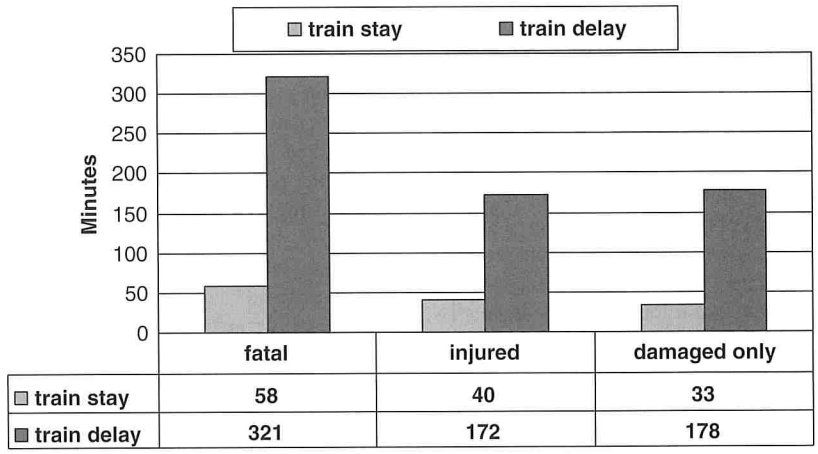


FIGURE 4 Average train stay and delay of traffic collisions at HRGXs.

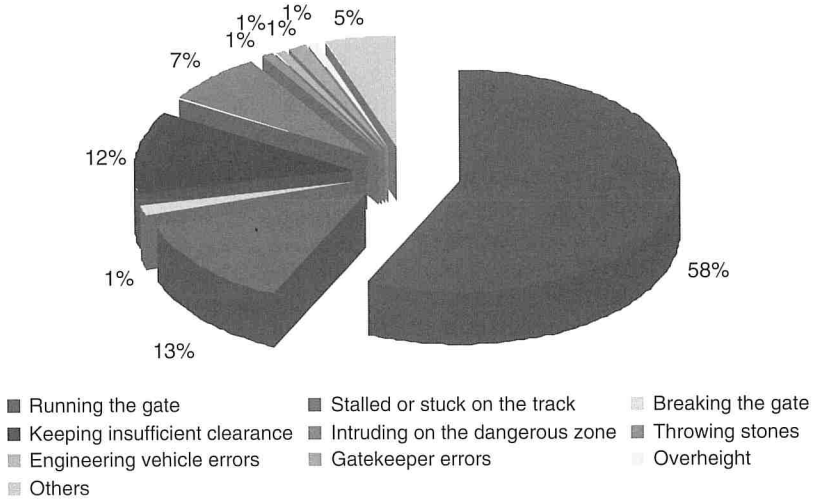


FIGURE 5 Causes of HRGX crashes in Taiwan.

determined. As a result, one could expect that a grade crossing with a shorter maximum TTS might result in a more difficult situation for a highway user to escape from the hazardous zone when a train is coming. A higher traffic collision frequency or rate would be observed at those HRGXs located near a curved segment with a small radius, which has a relatively short sight distance from the locomotive engineer's perspective.

The sight distance from the rail side is defined as the train's traveling distance between the point where the locomotive engineer initially sees the grade crossing (or an obstacle at the crossing) and the spatial location of the grade crossing. This specific distance could be converted into an equivalent time factor, defined as the time to train arrival (TTA), by assuming a fixed train speed, and it is easily measured by taking a locomotive. In contrast, the actual period from the moment the train starts to brake to the moment it strikes an object at a grade crossing is defined as time to collision (TTC). In practice, TTC is difficult to measure because traffic collisions at HRGXs are rare incidents and it is not easy to objectively find the initial braking time. In general, the TTC is highly dependent on the TTA or clearance time, which is also called the time to line crossing, of highway users (*9*). Jensen applied these indices to analyze relevant human factors in a grade-crossing crash incident and to investigate the effects of safety measures at passive HRGXs (*9*). The TTA would be a good estimate of the TTC and it could be used to predict crash frequency at a grade crossing.

The TTA depicts the time during which a highway user may or may not have sufficient time to escape from the dangerous zone. Theoretically, a shorter sight distance or smaller TTA at a grade crossing usually results in a higher possibility of a traffic collision. Thus, an investigation into the causal relationship between TTAs and crash frequencies at HRGXs becomes crucial to identifying the key risk factors for future safety-improvement purposes.

State-of-the-art methods for modeling traffic collisions or risk assessment at HRGXs range from statistical sampling techniques that use a hazard index in ranking hazardous HRGXs as candidates for grade separation (*10*) to count-data statistical inference by means of the Poisson or negative binomial (NB) models (*11, 12*) or their variants (e.g., the heterogeneous NB model or the Poisson lognormal model) to identify highway or HRGX black spots (*12–14*). In view of the unique capability of the zero-inflated model to capture events with extra zeros (i.e., excess number of sites with zero traffic collisions), such as the zero-inflated Poisson (ZIP) or zero-inflated NB regression model, Nam and Lee analyzed the Korean HRGX crash-frequency data by using a ZIP regression model (*15*). The zero-inflated count-data statistical models generally outperform the Poisson or NB models from the goodness-of-fit perspective. However, the ZIP model should be used with caution. Specifically, one needs to verify a logical basis for the dual-state process and clear boundary conditions between the two if models based on zero inflation are considered appropriate. Because the zero or safe state has a long-term mean equal to zero, ZIP models cannot properly reflect the process that generates crash data (*16, 17*). The Poisson regression model, in contrast, cannot handle over- and underdispersion data, which are negatively influenced by the low sample mean and small sample size bias, while the gamma function model is disadvantaged in predicting traffic collisions at grade crossings by the application of a short-term data set (*18*).

The purposes of this research are twofold. First, a stratified regression model is developed to analyze the causal relationships between crash frequency and a set of TTAs at HRGXs. Second, the developed model is applied to investigate crash likelihood under different combinations of grade-crossing attributes and other external factors. The empirical study results are summarized for future crash-prevention purposes.

The paper is organized as follows. The section on data description describes the data set used for the empirical study in which both historical crash data and grade-crossing inventory data were collected and analyzed. The section on methodology depicts the stratified regression model adopted in the causal analysis. The section on results and discussion provides the empirical study results and discusses policy implications. Finally, the section on conclusions and recommendations summarizes this paper and gives research conclusions and future study directions.

DATA DESCRIPTION

Crash and Grade-Crossing Inventory Data

TRA's database contained more than 20 years of grade-crossing crash data with detailed information about each crash, including location, time of occurrence, train number and type, highway user involved, and impact severity. In the empirical study, except for a few railway segments in the port and rural areas, data for 1,245 traffic collisions at grade crossings during the 14-year period from 1997 to 2010 were collected and analyzed. Furthermore, on average every 5 years, TRA publishes a grade-crossing inventory handbook containing information related to grade crossings, including milepost, number of tracks, crossing type, crossing width, and highway function. The information contained in the grade-crossing handbooks published in 1988, 2003, and 2008 was adopted in the empirical study.

In the past decade, the number of HRGXs has been gradually decreasing as a result of grade separation or highway rerouting. In addition, some of the crossing attributes, such as variables related to traffic exposure (e.g., highway vehicular traffic volume and daily trains) are not easy to collect. Therefore, only a few key factors were incorporated into the modeling process, and they were classified into two categories. The first category covers the geographical attributes of the crossing area, such as at or not at station, near or not near an intersection, and single or double tracks. The second category covers crash-related characteristics, including train type, rush hour, time of day, crash severity, and vehicle or individual involved in a crash. Both geographical attributes of the crossing area and actual crash characteristics were adopted to construct various classified regression models and are discussed here.

Geographical Attributes of Crossing Area

Scenario 1: at station. A grade crossing located within a distance of 20 chains (365.7 m) from the near side of a station platform is considered to be at station (*19*).

Scenario 2: near intersection. The MUTCD suggests that preemption should be applied when the distance between a signalized intersection and an HRGX is less than 60 m (*8*). However, if a signal preemption scheme is needed because of a traffic situation, then the distance between an HRGX and the signalized intersection should be disregarded (*20*). A preempted HRGX with an intersection(s) nearby is defined as near intersection.

Scenario 3: single track. The lines in the eastern coast rail corridor and the branch lines serve smaller passenger and freight volumes than do those in the western main rail corridor; they are currently operating on a single-track rail infrastructure.

Crash-Specific Characteristics

- Train type. The TRA runs three passenger train types: (*a*) push–pull electric locomotives at the front and end, with some passenger cars in between (PP train, the first-class express train), (*b*) electric locomotive with some passenger cars behind (EL train, the second-class express train), and (*c*) electric multiple unit (EMU train, the local train). The EL train consists of one engine (locomotive) and several coaches. The EMU train is similar to a mass rapid transit train where multiple pairs of electric-power car units are chained.
- Peak hour. Three traffic peak periods of the highway were observed, namely 7:00 to 9:00 a.m., 11:00 a.m. to 1:00 p.m., and 5:00 to 7:00 p.m., and these periods were defined as the peak hours in a day. The rest of the time in a day was classified as the off-peak hours.
- Daytime. Daytime was defined as the duration from 6:00 a.m. to 6:00 p.m., and the rest of the hours in a day were considered the nighttime period.
- Crash severity. The levels of crash severity at an HRGX were defined as fatality, injury, and damage only. These levels are commonly used for the evaluation of the impacts of a grade-crossing crash, both from the individual and societal perspectives.
- Struck individual or vehicle. According to the physical characteristics of individuals or vehicles struck in a traffic collision at an HRGX, the struck highway users were classified into four groups: pedestrian (including bicycle), motorcycle or motor scooter, car (including van and small truck), and truck (including bus and trailer).

TTA Data Collection

A field survey lasting for more than 2 years was conducted to collect the TTAs of each grade crossing. Researchers estimated the TTAs by checking a stopwatch while they traveled around the TRA system on different locomotives. The TTAs associated with a specific grade crossing were generally measured by six unique values, because each grade crossing is typically traversed by three types of trains in two opposite railroad directions. Some of the investigated grade crossings might have only two or four TTA values, depending on how many types of trains traversed these specific grade crossings. For example, only two types of trains run on the branch lines of the TRA system; a total of four TTA values would be measured for these grade crossings along the branch lines in both directions. To obtain more accurate TTA values for each type of train, the specific TTAs of a particular train were measured at least twice or three times in one railroad direction at the grade crossing being surveyed; average values were taken for the final TTAs. The TTAs associated with a total of 479 grade crossings were respectively surveyed and estimated. Among the collected TTAs, the shortest TTA of 5 s was found on a PP train going through the shortest radius of a specific railroad curve. TTAs beyond 22 s were not taken into consideration or used in the modeling process, because for such a long sight distance generally observed in a tangent rail segment, most of the trains can come to a complete stop in the case of an emergency. Finally, all 479 grade crossings being investigated were the active type in which warning bells, flashing lights, and auto barrier gates were installed. Therefore, the developed model did not consider active warning versus passive warning when determining sight distance.

METHODOLOGY

The CCY–TTA data associated with the three geographical scenarios for the crossing area were analyzed by a general classification regression model. Generally, a grade crossing might or might not be located at a station (denoted as at station or not at station), might be close to a highway intersection or not (denoted as near intersection or not near intersection), and have a single track or double tracks (denoted as single track or double track). These geographical or location-specific attributes of a grade crossing are the potential engineering-based countermeasures that could be pursued. Each factor associated with the three geographical scenarios for the crossing area was paired with five collision-specific scenarios, including (*a*) type of train, (*b*) daytime or nighttime, (*c*) peak hour or off-peak hour, (*d*) severity level, and (*e*) type of highway user. The corresponding TTAs and CCYs for each paired combination of the crossing-area geographical factors and collision attributes were formulated with a nonlinear regression model to investigate the causal relationships between CCYs and TTAs. When the stratified combination of the covariates was used for the development of a regression-based model, the significant crash influence factors can be specifically identified. Furthermore, such a general classification regression model could avoid the collinearity problem possibly encountered with the use of a linear regression model.

Theoretically, CCYs monotonically decrease with an increase of TTAs under different combinations of the crossing-area spatial locations and crash attributes, because highway users have a higher probability of escaping from the danger zone with a larger TTA. In other words, the TTA, which implicitly represents the sight distance from the railway side, is expected to be negatively correlated with the crash frequency or collision rate of a grade crossing.

To investigate the causal relationships between CCYs and TTAs for the data collected at the 479 grade crossings, the following regression-based models were developed: (*a*) the general model, which formulates the causal relationship between the CCY and TTA data of all grade crossings, and (*b*) the classified regression models for each subset of the CCYs and TTAs associated with the specific grade crossings stratified by different combinations of crossing-area geographical attributes and crash characteristics. Data for calibrating the two nonlinear regression models were prepared separately. The formulations for both models are given in Equations 1 and 2:

$$CCY_{1i} = \beta_{10} + \beta_{11}TTA_i + \beta_{12}TTA_i^2 + \varepsilon_{1i} \quad (1)$$

$$CCY_{2i} = \beta_{20} + \beta_{21}TTA_{ij} + \beta_{22}TTA_{ij}^2 + \varepsilon_{2i} \quad (2)$$

where

CCY_{1i}, CCY_{2i} = average collision rates of the *i*th crossing for the general model and classified model, respectively;
TTA_i, TTA_{ij} = TTAs for the *i*th crossing and for the *i*th crossing under the *j*th stratified combination of the covariates, respectively;
β_{10}, β_{11}, β_{12} = coefficients to be estimated for the general model;
β_{20}, β_{21}, β_{22} = coefficients to be estimated for the classified model; and
ε_{1i}, ε_{2i} = error terms in the general model and classified models, respectively.

In the empirical study, the TTA and CCY data associated with different combinations of the three crossing-area geographical scenarios and five crash-related scenarios were analyzed with a nonlinear

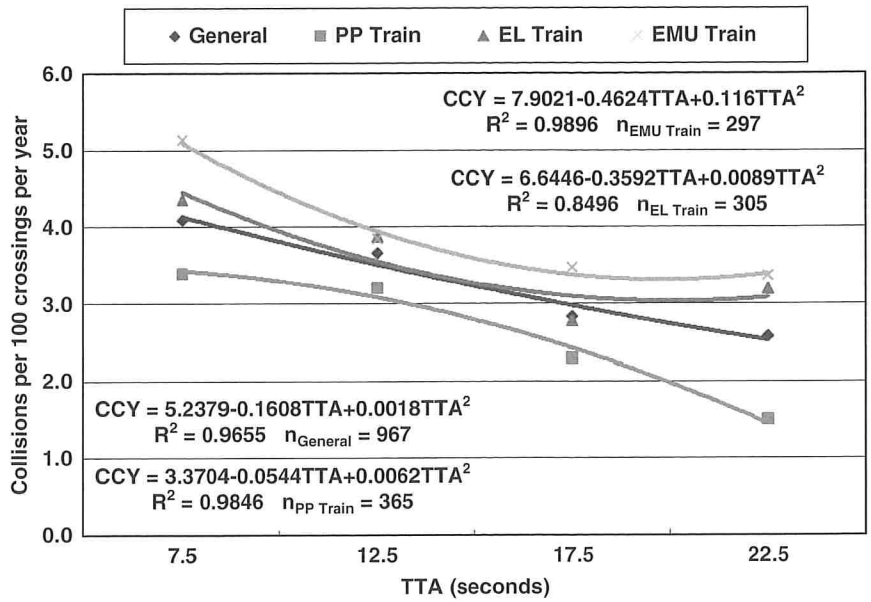


FIGURE 6 Parabolic regression models (both general and train type).

classification regression model in a parabolic form. Sensitivity analysis on the change of the TTA on the CCY was also conducted to investigate the key risk factors that might potentially contribute to traffic collisions at grade crossings. Finally, the developed models were applied to predict crash rates to foresee the potential impacts on increasing or decreasing TTAs.

RESULTS AND DISCUSSION

Relationships Between CCYs and TTAs

The classified CCY–TTA models for various train types and crossing-area geographical attributes were calibrated and are respectively shown in Figures 6 and 7. Also shown in these two figures are the model calibration results of the general models with the CCY–TTA data of the total 479 grade crossings. Based on the experimental setups and model calibration results, the findings of the empirical study are as follows:

- Both the general regression model and the three classified regression models of different train types or crossing-area geographical locations appear to have a significantly negative trend between the CCYs and TTAs (see Figures 6 and 7), which indicates that the shorter the TTA, the higher the CCY.
- The model calibration results for individual classified regression models are shown in Table 1. Again, most of these results indicate a significantly or slightly significantly negative trend between CCYs and TTAs. However, some empirical study results contradicted what was expected. For instance, the model calibration results for the

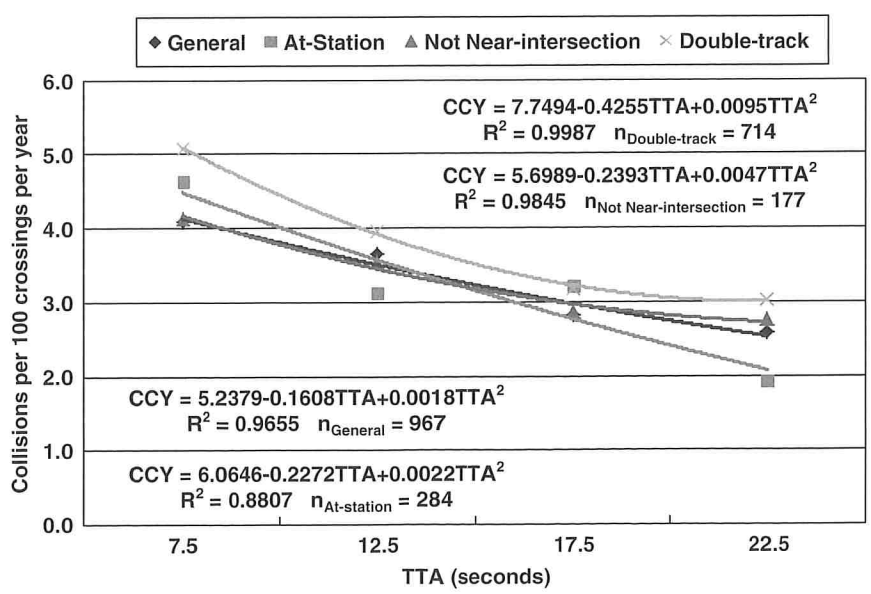


FIGURE 7 Parabolic regression models (both general and crossing location).

TABLE 1 Regression Trends

Characteristic	At Station	Not at Station	Near Intersection	Not Near Intersection	Single Track	Double Track
PP train	SN	SN	SN	SP	—	SN
EL train	SN	SN	SN	LP	SP	SN
EMU train	SN	SN	LN	SN	LP	LN
Peak hours	SN	—	—	SN	SP	SN
Off-peak hours	SN	SN	SN	SP	SP	SN
Nighttime	SN	SN	SN	SP	SP	SN
Daytime	LN	SN	SN	SN	SP	SN
Fatality	SP	SN	—	SN	SP	SN
Injury	—	SN	SN	SP	LN	SN
Damage only	SN	LN	SN	SN	—	SN
Pedestrian	SP	—	LP	SN	LP	—
Motorcycle	—	SN	LN	LN	SP	SN
Car	SN	SN	SN	SP	SP	SN
Truck	SN	—	LN	NA	NA	LN

NOTE: SN = significantly negative relationship between CCYs and TTAs; LN = slightly significantly negative relationship between CCYs and TTAs; SP = significantly positive relationship between CCYs and TTAs; LP = slightly significantly positive relationship between CCYs and TTAs; NA = no data available; — = no significantly positive or negative relationship between CCYs and TTAs.

CCY–TTA data related to single tracks exhibited no significantly causal relationships or even a positive causal relationship between the CCYs and TTAs. There may have been insufficient data, or the very small number of traffic collisions observed at these single-track HRGXs (generally located at branch lines with relatively less daily train service or highway vehicular traffic) may have affected the results.

- In the analysis of Scenario 1 (at station or not at station), for HRGXs close to a railway station (at station), a relatively higher collision rate was observed for highway users of cars or trucks and during peak hours or nighttime. However, for HRGXs far from a railway station (not at station), more severe traffic collisions in terms of fatalities or injuries were found, and a higher collision rate was encountered for highway users of motorcycles or cars and during off-peak hours.
- In the analysis of Scenario 2 (near intersection or not near intersection), for HRGXs preempted with a highway intersection(s), a relatively higher traffic collision rate was found for highway users of cars or trucks, PP or EMU trains, and during off-peak hours. However, for HRGXs far from a highway intersection (not near intersection), more fatal traffic collisions were found, and a higher collision rate was found for pedestrians and highway users of motorcycles, the EMU train, and during peak hours or daytime.
- In the analysis of Scenario 3 (single track or double track), most classified regression models related to double tracks revealed a significantly or slightly negative relationship between CCYs and TTAs. Such a negative trend was not found in the models related to single tracks, because the CCY and TTA data on grade crossings located at the single-track infrastructure were very limited. An additional finding is that motorcyclists constituted the most dangerous highway user group at a double-track HRGX area.

Table 2 illustrates the effects of changing TTAs on the change of CCYs. As indicated in the sensitivity analysis results shown in Table 2, a PP or EL train running on double-track rail lines, or close to a station (at station), or near a highway intersection (near intersection) would benefit the most decreasing traffic collision rates realized by appropriately increasing the TTAs or the sight distance from the railway side. Countermeasures aimed at improving safety levels related to damage-only crashes at stations is another priority that is worthy

TABLE 2 Sensitivity Analyses

Characteristic	At Station	Not at Station	Near Intersection	Not Near Intersection	Single Track	Double Track
PP train	–0.1115	–0.0175	–0.1321	0.2823	0.0555	–0.1060
EL train	–0.7792	0.0816	–0.1768	0.3341	0.3151	–0.3312
EMU train	–0.2322	–0.2629	–0.1513	–0.9117	0.2698	–0.3821
Peak hours	–0.1361	0.0350	0.0039	0.0641	–0.0032	0.0152
Off-peak hours	–0.2042	–0.0896	–0.1199	0.0168	0.2529	–0.2396
Nighttime	–0.1390	0.0622	–0.0262	0.1801	0.2339	–0.0640
Daytime	–0.0024	–0.0768	–0.0932	–0.0992	0.0349	–0.1604
Fatality	0.0627	–0.0684	–0.0485	–0.0549	0.0878	–0.1097
Injury	0.0092	–0.0452	–0.0868	0.1450	0.1376	–0.1008
Damage only	–0.3349	0.0997	0.0102	–0.0259	0.0169	–0.0138
Pedestrian	0.1855	0.0018	0.0472	–0.0076	0.0571	0.0168
Motorcycle	–0.0482	–0.0603	–0.0482	–0.1175	0.0614	–0.1095
Car	–0.2000	0.0157	–0.0760	0.1435	0.1005	–0.0826
Truck	–0.1484	–0.0039	–0.0365	0.0031	0.0042	–0.0403

NOTE: Sensitivity = $\Delta CCY/\Delta TTA$.

TABLE 3 Predicted Crash Frequencies for Different TTAs and Influence Factors

TTA	General	Train			At Station	Not Near Intersection	Double Track
		PP	EL	EMU			
5.0	4.48	3.49	5.07	5.88	4.98	4.62	5.86
6.0	4.34	3.47	4.81	5.55	4.78	4.43	5.54
7.0	4.20	3.45	4.57	5.24	4.58	4.26	5.24
8.0	4.07	3.41	4.34	4.95	4.39	4.09	4.95
9.0	3.94	3.36	4.13	4.68	4.20	3.93	4.69
10.0	3.81	3.29	3.95	4.44	4.01	3.78	4.45
11.0	3.69	3.22	3.77	4.23	3.83	3.64	4.22
12.0	3.57	3.13	3.62	4.03	3.66	3.51	4.01
13.0	3.45	3.03	3.48	3.86	3.48	3.39	3.83
14.0	3.34	2.92	3.37	3.71	3.32	3.28	3.66
15.0	3.23	2.79	3.27	3.59	3.15	3.18	3.51
16.0	3.12	2.65	3.18	3.49	3.00	3.09	3.38
17.0	3.02	2.50	3.12	3.41	2.84	3.00	3.27
18.0	2.92	2.34	3.07	3.35	2.69	2.93	3.18
19.0	2.83	2.16	3.04	3.32	2.55	2.87	3.10
20.0	2.74	1.98	3.03	3.31	2.41	2.81	3.05
21.0	2.65	1.78	3.04	3.33	2.27	2.77	3.01
22.0	2.57	1.56	3.06	3.37	2.14	2.73	3.00

NOTE: Crash frequencies shown as CCYs.

of further pursuit. The results of the sensitivity analysis provide the government agency or rail operators with beneficial information to prioritize monetary resources for safety improvement under budgetary constraints.

Model Application

With the developed classified regression models, traffic collision rates under different train types and crossing-area geographical attributes were predicted; the results are shown in Table 3. In the 5- to 22-s interval of TTAs, the CCYs of the two train types (including the EL train and the EMU train) and the double tracks are greater than those of the general regression model, which means that the collision rates of these two trains or CCYs at double-track grade crossings are higher than the average values. The predication capability of the developed nonlinear regression models could foresee the crash probabilities under different grade-crossing attributes, traffic conditions, or both.

CONCLUSIONS AND RECOMMENDATIONS

With the use of a classified regression modeling approach, this research investigated the effects of TTAs on traffic collision rates. In particular, the causal relationships were analyzed under different combinations of crash and crossing-area geographical attributes. This research aimed at developing a regression-based risk model to conduct a causal analysis of how to better evaluate risk levels at HRGXs.

In the numerical analysis, the TTA was shown to be significantly and negatively associated with the traffic collision rate at HRGXs. To reduce crash frequency that is due to a short TTA or sight distance, railway agencies or operators might consider means aimed at increasing the sight distance of the locomotive engineer at a rail curve segment or rerouting highways to relocate the HRGX from a curve segment to a tangent one. Alternatively, the railway authority might install advanced obstacle detection devices at HRGXs to provide the locomotive engineer with advanced warning information so that some emergency-appropriate actions could be taken in a timely manner.

In the experimental setup, only three scenarios of the crossing-area attributes were classified for the development of the CCY–TTA model. The proposed model can be modified to explore the potential effects of other relevant crossing-area geographical attributes and traffic controls, such as crossing angle, slope, width, and warning devices. In addition, because the TRA has maintained a good database on train holding and delay data on traffic collisions at HRGXs, one of the future directions for research that could be realized soon is the analysis of social cost induced by traffic collisions at HRGXs in light of the proposed nonlinear regression model framework. Finally, obstacle detection devices and video recording systems are installed at some grade crossings in Taiwan; another research topic would be on investigation of the effect of the obstacle detection device or video system on crash prevention at HRGXs.

ACKNOWLEDGMENTS

The authors thank TRA, which provided the historical crash data and grade-crossing inventory data set in the empirical study. This research was partially supported by a grant from the National Science Council, Taiwan.

REFERENCES

1. Mineta, N. *Secretary's Action Plan on Highway–Rail Crossing Safety and Trespass Prevention.* U.S. Department of Transportation, Washington, D.C., 2004.
2. Davies, L. *Annual Safety Performance Report 2006.* Rail Safety and Standards Board, London, 2007.
3. Department of Statistics. *Analysis of Traffic Accidents for the Taiwan Railway System.* Ministry of Transportation and Communications, Taiwan, 2007.
4. Taiwan Railways Administration. *Annual Railroad Safety Report in Taiwan.* Ministry of Transportation and Communications, Taiwan, 2011.
5. Berg, W. D., K. Knoblauch, and W. Hucke. Causal Factors in Railroad–Highway Grade Crossing Accidents. In *Transportation Research Record 847,* TRB, National Research Council, Washington, D.C., 1982, pp. 47–54.
6. Gou, M., and O. Bellavigna-Ladoux. Impact of Heavy Vehicles on Crossing Safety—Development of an Adapted Design Tool. Publication TP 14172E. Transport Canada, Ottawa, Ontario, 2003.
7. Ogden, B. D. *Railroad–Highway Grade Crossing Handbook,* rev. 2nd ed. DTFH61-00-C-0003. FHWA, U.S. Department of Transportation, 2007.
8. *Manual on Uniform Traffic Control Devices,* millennium ed. FHWA, U.S. Department of Transportation, 2000.
9. Jensen, S. U. *Passive Railroad–Highway Grade Crossings: Relevant Factors and Effects of Safety Measures.* Technical Report, Trafitec, Denmark, 2007.
10. Gitelman, V., and A. S. Hakkert. The Evaluation of Road-Rail Crossing Safety with Limited Accident Statistics. *Accident Analysis and Prevention,* Vol. 29, No. 2, 1997, pp. 171–179.
11. Austin, R. D., and J. L. Carson. An Alternative Accident Prediction Model for Highway–Rail Interfaces. *Accident Analysis and Prevention,* Vol. 34, No. 1, 2002, pp. 31–42.
12. Saccomanno, F. F., L. Fu, and L. F. Miranda-Moreno. Risk-Based Model for Identifying Highway–Rail Grade Crossing Blackspots. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1862,* Transportation Research Board of the National Academies, Washington, D.C., 2004, pp. 127–135.
13. Saccomanno, F. F., R. Grossi, D. Greco, and A. Mehmood. Identifying Black Spots Along Highway SS107 in Southern Italy Using Two Models. *Journal of Transportation Engineering,* Vol. 127, No. 6, 2001, pp. 515–522.
14. Miranda-Moreno, L. F., L. Fu, F. F. Saccomanno, and A. Labbe. Alternative Risk Models for Ranking Locations for Safety Improvement. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1908,* Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 1–8.
15. Nam, D., and J. Lee. Accident Frequency Model Using Zero Probability Process. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1973,* Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 142–148.
16. Lord, D., S. P. Washington, and J. N. Ivan. Poisson, Poisson-Gamma and Zero-Inflated Regression Models of Motor Vehicle Crashes: Balancing Statistical Fit and Theory. *Accident Analysis and Prevention,* Vol. 37, No. 1, 2005, pp. 35–46.
17. Lord, D., S. P. Washington, and J. N. Ivan. Further Notes on the Application of Zero-Inflated Models in Highway Safety. *Accident Analysis and Prevention,* Vol. 39, No. 1, 2007, pp. 53–57.
18. Lord, D., and F. Mannering. The Statistical Analysis of Crash-Frequency Data: A Review and Assessment of Methodological Alternatives. *Transportation Research Part A: Policy and Practice,* Vol. 44, No. 5, 2010, pp. 291–305.
19. Smith, A. *Level Crossing Safety Performance Report, June 2006.* Rail Safety and Standards Board, London, 2006.
20. Cho, H., and L. R. Rilett. Improved Transition Preemption Strategy for Signalized Intersections Near At-Grade Railway Grade Crossing. *Journal of Transportation Engineering,* Vol. 133, No. 8, 2007, pp. 443–454.

The Highway–Rail Grade Crossings Committee peer-reviewed this paper.

Evaluating Safety at Railway Level Crossings with Microsimulation Modeling

Li-Sian Tey, Inhi Kim, and Luis Ferreira

Safety at railway level crossings (RLXs) is a worldwide issue that increasingly attracts the attention of relevant transport authorities, the rail industry, and the general public. The differences in the operation characteristics of varying types of warning devices, together with differences in crossing geometry, traffic, or train characteristics, leads to different driver behaviors at crossings. The aim of this study was to use traffic microsimulation modeling based on field video recording data to compare the safety performance of varying conventional RLX warning systems. The widely used microsimulation model VISSIM was modified to produce safety-related performance measures, namely, collision likelihood, delay, and queue length. The results showed that RLXs with an active warning system were safer than those with a passive sign by at least 17%. Integration of surrogate measures in conjunction with traffic simulation models determined which safety approach was more efficient for specified traffic and train volumes.

Railway level crossings (RLXs) present serious potential conflicts between road vehicles and trains that can produce severe traffic outcomes. Safety at RLXs is a worldwide issue that increasingly attracts the attention of transport authorities, the rail industry, and the general public. More than 2,000 accidents occurred at active and passive railway crossings in the United States each year from 2006 through 2010 (*1*). Each year, hundreds of people across Europe die in accidents at level crossings; these deaths account for one-third of all rail fatalities and 1% to 2% of all road deaths (*2*). In Australia (excluding New South Wales), 366 fatalities and 909 serious personal injuries resulted from 654 road vehicle collisions at RLXs between 2001 and 2009 (*3*).

Like many countries, Australia has seen an improvement in terms of safety at RLXs as active or automated systems have replaced passive warning systems (*4, 5*). Studies show that driver behavior responses are different at RLXs with different control devices (*6, 7*). Active or automated systems promote safer driver behavior. However, upgrading conventional active warning systems at RLXs is costly, especially in regional areas where train frequency and roadway traffic volumes are low. Furthermore, vehicle collisions at crossings are reported to be mainly attributed to driver behavior (i.e., noncompliance) in response to the warning device (*8*). Therefore, evaluating the safety of different types of warning systems at RLXs is of importance.

Faculty of Engineering, Architecture, and Information Technology, University of Queensland, Brisbane St. Lucia, Queensland 4072, Australia. Other affiliation for L.-S. Tey: Faculty of Civil Engineering, Universiti Teknologi MARA, 40450 Shah Alam, Selangor, Malaysia. Corresponding author: I. Kim, inhikim@uq.edu.au.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 70–77.
DOI: 10.3141/2298-08

This paper evaluates and compares the safety performance of the various existing RLX warning systems. Traffic microsimulation modeling was used to collect field data from the selected sites. The use of microsimulation with surrogate measures provides an enhanced way to conduct safety evaluation without risk. The use of traffic simulation for safety assessment usually confronts some difficulties, because traffic simulation assumes, based on car-following theory, that each vehicle has a gap in terms of distance. Some adjustments have been implemented to overcome this issue. This study includes three indicators to evaluate safety: the potential number of conflicts, delay, and queue length. The paper is structured as follows. The section on past work critically reviews traffic conflict techniques in traffic microsimulation modeling; the section on the development of the simulation model provides a brief description of the procedure and results of the field data collection (as input for model development in the next section); and the section on simulation results sets out the model development steps to present and compare the results. The final section concludes with the main findings.

PAST WORK

Safety-Related Issues

Accident data are traditionally gathered through police records. Safety assessments of traffic systems are usually analyzed on the basis of particular police-reported historical accident data. Therefore, safety assessments are usually reactive. However, it is unreasonable to wait for a large enough sample size (accidents) to occur before traffic systems can be evaluated, especially at RLXs, where police-reported historical records may not be sufficient for analysis either quantitatively or qualitatively (collisions at RLXs are rare but catastrophic). To address these constraints, traffic conflict techniques provide the possibility of modeling driver variation and proactive measurements.

The concept of traffic conflicts was first proposed by Perkins and Harris as an alternative to accident data, in relation to evasive maneuvers taken by drivers (*9*). In the early studies, an assumption that the conflicts must exist before crashes occurred was widely accepted to define the conflicts. However, in many cases, the accuracy of this assumption was doubtful (*10, 11*). At the First Workshop on Traffic Conflicts, an enhanced definition was suggested and agreed upon. Thereafter, the internationally accepted definition of a traffic conflict became (and remains) "an observable situation in which two or more road users approach each other in space and time to such an extent that there is a risk of collision if their movements remain unchanged" (*12*). This alternative definition has been adopted in a number of safety studies (*13–17*). Sayed and Zein developed predictive models that use linear regression analysis to relate the number

of traffic conflicts to traffic volume and accidents (*18*). Sayed et al. related the traffic conflict technique to traffic microsimulation for safety assessment at unsignalized intersections (*19*). Archer regarded the traffic conflict technique as the most developed indirect measure of traffic safety (*20*).

Speed, as one of the fundamental characteristics of traffic flow, is commonly used to measure traffic performance on highway systems. It is also used as an indication of level of service in accident analysis, and in economic studies (*21*). Anderson and Krammes, in their research analyzing the statistical relationship between mean accident rate and mean speed reduction, suggested that estimated speed reduction is a useful measure that helps explain how an accident experience at horizontal curves on rural two-lane highways varies with the degree of curvature (*22*). Noncompliance refers to nonadherence to the traffic rules or traffic control devices (i.e., running a red light, not stopping at a stop sign, etc.). A study by Abraham et al. tested the relationship between driver violations and RLX collisions (*23*). It revealed the potential of using violation data to determine the relative hazardousness of highway–rail crossings in combination with crash histories. The violation data may also be used to develop countermeasures that would help alleviate violations and eventually traffic collision problems at RLXs. Lee emphasized that delay was one of the prime indications for evaluating RLXs because the crossings might create excessive delays for road traffic and therefore possible violations (*24*). He concluded that a more detailed analysis had to be performed if the average delay exceeded a certain level.

Application of Traffic Microsimulation in Evaluation of Safety Measures

Traffic microsimulation models play a pivotal role in assessing current traffic conditions and then analyzing, assessing, and evaluating traffic problems. They allow alternative traffic operation systems to be tested in a simulated environment without physical interruption of existing traffic networks. This approach provides a platform for a performance comparison of varying solutions for decision making. It is a cost-effective approach for accessing implications of alternative traffic systems.

The use of traffic microsimulation for safety assessment usually encounters some difficulties because the microsimulation assumes that all road users drive their vehicles in a safe manner. Most of the commercially available microsimulation models are designed to evaluate traffic efficiency rather than traffic safety. Safety-related behavior assumptions in most of the traffic microsimulation models are tailored to represent rule-adherent drivers, which may not replicate real-world situations (e.g., speeding or red light running).

Therefore, traffic microsimulation models have a number of restrictions for traffic safety analyses. This limitation is mainly attributed to the high degree of variance in driver perception, reaction, and driving behavior and errors. These variations in human factors create potential conflicts such as incidents, near misses, and collisions. In addition, traffic microsimulation models are designed to avoid an occurrence of traffic collisions. In other words, all the driver–vehicle units in the traffic microsimulation will comply with traffic regulations (preset traffic microsimulation rules). However, this simulated situation does not exist in reality. For instance, not all motorists adhere to traffic control devices at RLXs (*6*).

As pointed out by Archer and Kosonen, the relatively strict deterministic rule base that governs this type of simplified driver behavior must be loosened to allow for a realistic amount of behavioral variance and a possibility for errors to occur (*25*). These limitations have been addressed by some researchers (*14, 20, 26, 27*). Gettman and Head evaluated important parameters in characteristics against commercially available traffic microsimulation models (*28*). All of the reviewed models required some level of modification, upgrade, or enhancement to support the derivation of surrogate safety measures. It does not appear possible to obtain surrogate measures from any of the reviewed traffic microsimulation models without some internal modifications to either the application programming interfaces or the source code modules themselves.

Despite the limitations, the traffic microsimulation approach is recommended for safety assessment because it has several advantages:

- It allows a hazardous circumstance to be explored safely. It is difficult to conduct field research on the likelihood of a crash, because such research may endanger lives or interrupt traffic operations.
- The output obtained should be reliable if calibration of the simulation is done logically.
- It enables users to evaluate a result with a relatively small amount of time and money.
- It enables users to evaluate a benefit before a safety system is implemented.
- Many different scenarios can be tested.

Several approaches to traffic safety issues in traffic microsimulation have appeared in recent years. Although a significant number of studies have focused on road safety, none have involved RLXs (intersection collisions). Through the use of railway crossings as case studies, this paper attempts to apply a traffic conflict technique to traffic microsimulation. Three surrogate safety measures are adopted: compliance, delay, and queue length.

DEVELOPMENT OF SIMULATION MODEL

General Approach

VISSIM was selected as a traffic microsimulation tool for this study. It is a widely used software package for microscopic multimodal traffic flow simulation that was developed in Germany. The general form of the car-following model is based on the psychophysical driver behavior model developed by Wiedemann and Reiter (*29*). The basic concept of this model is that a faster-driving vehicle reduces its speed to that of a slower-driving vehicle as a driver's perception threshold is reached. The faster-driving vehicle decreases speed below the slower-driving vehicle until it can reaccelerate after the event. This iterative procedure remains until the simulation is finished (*30*).

In this study, a component object model interface developed in Visual Basic was integrated with VISSIM 5.3. VISSIM has been modified to produce safety measurements at RLXs with varying warning devices. Field data collected from three RLXs near Brisbane, Australia, were input to the model. The procedures of field data collection and the results are discussed in the next subsection. The outline of the model development is shown in Figure 1.

Model development involved two components: compliance (probability of stopping) and speed profile. The probability of stopping in this study was defined as the rate of driver compliance with the warning devices at RLXs. This component indicates whether or not drivers proceed across crossings when warnings are activated.

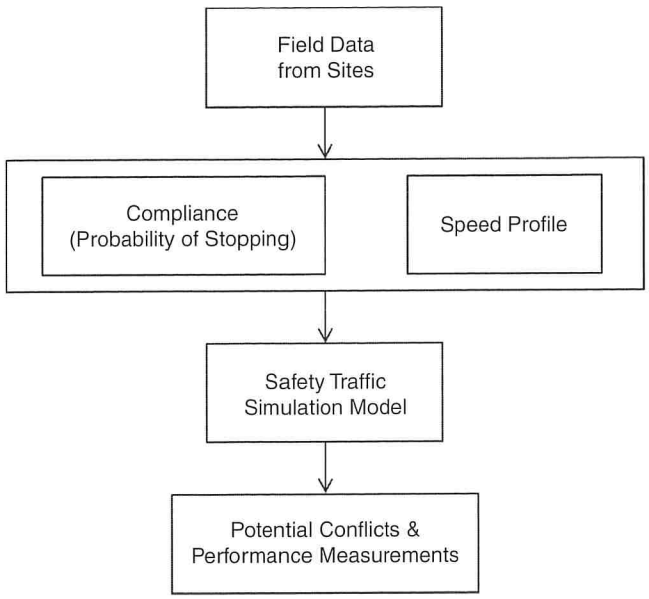


FIGURE 1 Outline of model development procedure.

In VISSIM, this is a part of the signal control function under the driving-behavior parameter sets. Speed profiles that were collected from the field data were used. These two analyzed components were the main factors that contributed to the development of the simulation model. A detailed explanation about these components is given in the following paragraphs.

Field Data Collection

Data were collected with a portable traffic surveillance camera. The selected study sites were carefully investigated so that the installed camera would be hidden from the view of the drivers, because their knowledge of the device could affect their driving behavior. The camera was erected near the crossing area to capture the operation of the warning devices as well as a roadway section of more than 200 m from the stop line. Once the recording process started, distance was measured. Steel plates painted in white were placed at 20-m intervals from the stop line (referred to as 0 m) to 200 m on both sides of the road.

Video footage was captured under normal daylight conditions from 6:00 a.m. to 5:30 p.m. at 25 video frames per second. Data were collected for all vehicle types, including passenger vehicles, trucks, and buses. From field observation during site selection trips, an approaching distance of 200 m was determined to be sufficient for drivers of all vehicle types to react appropriately to the warning devices. Because the major traffic composition at all three study sites was mainly passenger vehicles, data were not grouped into vehicle types.

The primary data retrieved were stopping compliance (violation) and approaching speed profile (*7*). The results summarized in Table 1 were input to the modified VISSIM model; Site 1 has a stop sign, Site 2 has flashing lights only, and Site 3 has a boom barrier with flashing lights. The data input included the stopping-compliance rate, the average speed recorded at the last two intervals (0 to 20 m and 20 to 40 m) of road section approaching an RLX, and the standard deviation of the averaged speed.

Driver stopping compliance is defined as total adherence to traffic rules. For instance, crossings with stop signs require a motorist to stop completely to look for the presence of a train in both directions. Similarly, motorists should stop when an active control system is activated by an approaching train. For Site 1, vehicles that did stop in front of the stop line were regarded as compliant; other movements such as slowing down and driving through were categorized as noncompliant. For Sites 2 and 3, stopping compliance was measured as soon as the warning devices were activated. The two reactions were categorized as stopped (compliance) or drove through (noncompliance). The observations show that the compliance rate at the passive crossing with stop sign (Site 1) was 41%, which is considerably lower than that at the two active crossings (70% at Site 2; 77% at Site 3). The compliance rate at Site 3 was slightly higher than that at Site 2. Similar results have been reported by various studies (*31–33*), although Abraham et al. found that drivers tended to commit more violations at the gated level crossings than they did at those that had only flashing lights (*23*). These differences exist as a result of different localized site conditions, driver behavior, and environmental conditions.

TABLE 1 Field Results for VISSIM Model Development

Input Datum	Site 3	Site 2	Site 1
Compliance rate (%)	77	70	41
Average speed (km/h)			
0–20 m	24.7	27.2	16.3
20–40 m	34.5	33.8	38.4
Standard deviation (km/h)			
0–20 m	18.9	19.9	6.4
20–40 m	16.2	11.3	9.6

Probability of Stopping (Driver Compliance)

At a crossing equipped with flashing lights only or with boom barrier with flashing lights, drivers tended to drive as if they were driving through a road intersection with an amber (yellow) signal. Although some drivers were willing to comply with flashing lights or boom gates, others were not patient enough to wait until a train passed the RLX. Because drivers make a decision either to go or stop, a probability model in VISSIM was used to replicate the stopping-compliance rate, as shown in Table 1. The probability model, p, is given by Equations 1 and 2:

$$p = \frac{1}{1 + \exp^{-ui}} \tag{1}$$

$$ui = \alpha + \beta_1 v + \beta_2 \mathrm{dx} \tag{2}$$

where

v = speed,
dx = distance, and
α, β_1, and β_2 = constants.

In VISSIM, speed and distance are variables used as a function of the probability of stopping at road intersections. In the study reported here, speed and distance were ignored (β_1, $\beta_2 = 0$) and α was replaced with the actual compliance rate from the field results because a major factor in this study is how warning systems themselves affect driving behavior. As the compliance rate of the RLX

with flashing lights, boom barrier with flashing lights, and stop signs is 70%, 77%, and 41%, α becomes 0.8, 1.1, and –0.4, respectively.

Safety Simulation Model

The signal control function in VISSIM has been modified to simulate a violation, which allows output of numbers of likelihood of collision. The developed simulation model contains train tracks, roads, types of vehicles, detectors, and signal controls as shown in Figure 2. Train tracks run east and west in both directions; roads intersect these train tracks in a vertical direction. In this study, the simulation model was designed for an area where trains run relatively less frequently (a suburban area at off-peak times). Trains arrived every 30 min and vehicles were introduced to the network at 450 and 900 vehicles per hour (vph), respectively. The simulation ran for 5 h (18,000 s) with intervals of 0.1 s. Three detectors played a role in triggering a virtual signal control so that warning devices were activated accordingly.

With these data as input for a traffic microsimulation model, an example network was developed to identify how many collisions were likely to take place at the RLX equipped with a stop sign, flashing lights, or boom barrier with flashing lights. The developed model assumed the following:

- A signal head function in VISSIM was adopted as stop line at the RLX.
- Vehicles moved in each direction and were simulated every second.
- Trains ran in each direction based on headway.
- The proportion of heavy goods vehicles was 10%.
- To reduce the theoretical deviations, the Monte Carlo method was applied.

Fifteen different random seeds were repeated to decrease uncertainty.

The following procedure is shown in Figure 2:

- When a train touched Detector 1, the signal head turned amber. In reality, RLX warnings are activated to approaching vehicles. Detector 1 was placed at the location where the warning was activated approximately 30 s before the train arrived at the crossing area; this procedure is in accordance with the Association of American Railroads manual, which is followed by the Australian Rail Track Corporation (*34*).
- Vehicles then followed the rule of the probability of stopping, which is derived from the compliance rate.
- If the vehicles proceeded to touch the area called likelihood of collision, then the simulation stored the vehicle identifier, speed, and vehicle type.
- When the train touched Detector 2, the vehicles completely stopped in front of the signal head. Detector 2 was placed at the location where, for the situations with the stop sign and the flashing lights, drivers could clearly see the train approaching but the boom barrier with flashing lights prohibited vehicles once they were fully closed.
- When the train touched Detector 3, the signal head turned green, allowing vehicles access to the RLX. Detector 3 was placed at the location where warning was deactivated.

SIMULATION RESULTS

When the RLX warning was activated by the train at Detector 1, detailed outputs from the VISSIM simulation, such as vehicle coding and the vehicle's speed, updating position, and acceleration, were extracted from the likelihood of collision on second-by-second and a vehicle-by-vehicle basis. Because there is a high possibility of a conflict at that point, vehicles involved in the likelihood-of-collision instances were recorded as potential conflicts.

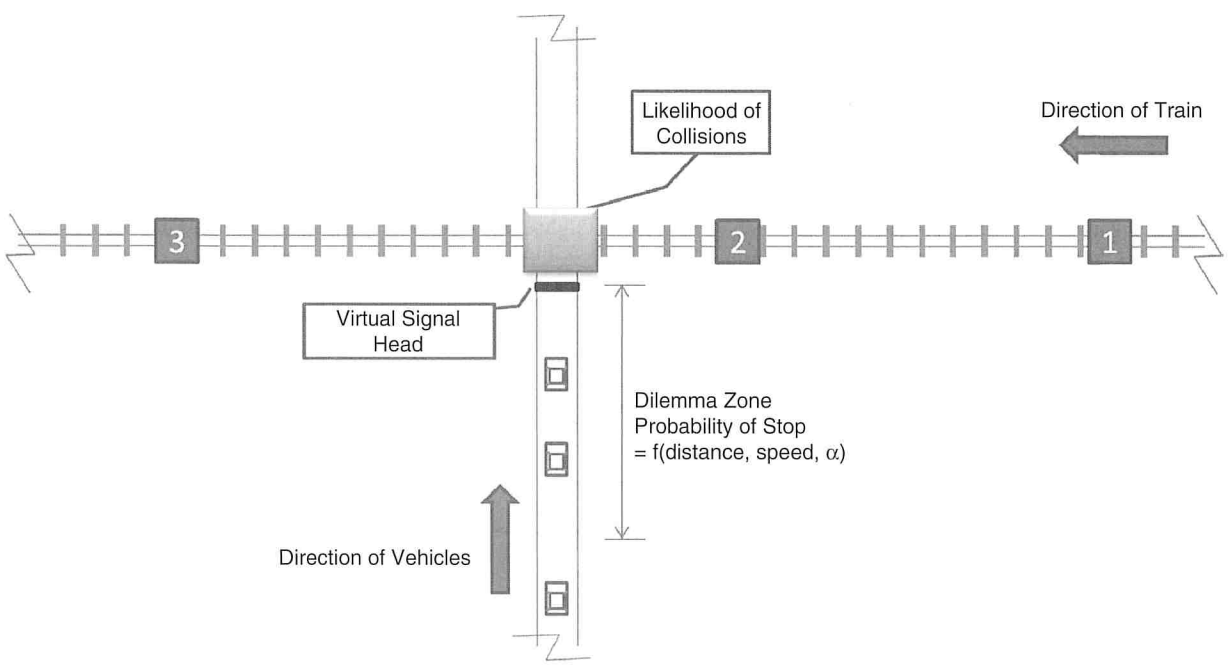


FIGURE 2 Layout of model in VISSIM.

TABLE 2 Number of Potential Conflicts

	Traffic Flow of 450 vph (each direction)			Traffic Flow of 900 vph (each direction)		
Seed	Site 3	Site 2	Site 1	Site 3	Site 2	Site 1
Run 1	10	13	37	7	13	74
Run 2	16	12	36	13	20	58
Run 3	19	21	33	11	14	38
Run 4	16	24	40	10	15	46
Run 5	15	20	41	12	17	55
Run 6	10	12	45	17	21	63
Run 7	12	15	59	13	17	55
Run 8	15	16	47	10	12	50
Run 9	9	9	39	17	18	48
Run 10	13	17	44	15	23	54
Run 11	11	18	55	9	13	43
Run 12	6	11	43	8	12	37
Run 13	6	7	30	9	10	32
Run 14	9	12	39	13	13	35
Run 15	6	10	42	14	15	34
Number of potential conflicts	12	14	42	12	16	48

Likelihood of Collision

The number of likely conflicts was compared at Sites 1 (stop sign), 2 (flashing lights), and 3 (boom barrier with flashing lights). Table 2 summarizes the results based on the number of trains and vehicles at the three RLXs. When the traffic flow was 450 vph in each direction and the train ran every 30 min, 14 potential conflicts were recorded at Site 2; in contrast, 12 potential conflicts were found at Site 3. In relative terms, there were 17% fewer potential conflicts at the crossing with a boom barrier with flashing lights than at the crossing with flashing lights only. When the traffic flow was increased to 900 vph in each direction, the potential conflicts recorded at Site 2 increased to 16, but the potential conflicts remained at 12 at Site 3. The results also show that Site 3 and Site 2 were safer than Site 1 by 300% and 350% to 400%, respectively.

As expected, the crossing equipped with the boom barrier with flashing lights (Site 3) had the least number of potential conflicts. When compared with the crossing with flashing lights only (Site 2), the difference was very small, but the crossing with the flashing lights had a high potential for conflicts. A comparison between different traffic volumes did not show significant relationships between warning systems. This is mainly because potential conflicts would occur for the first car approaching. If the first car stopped, then the cars following it had to stop behind it.

Delay and Queue Length

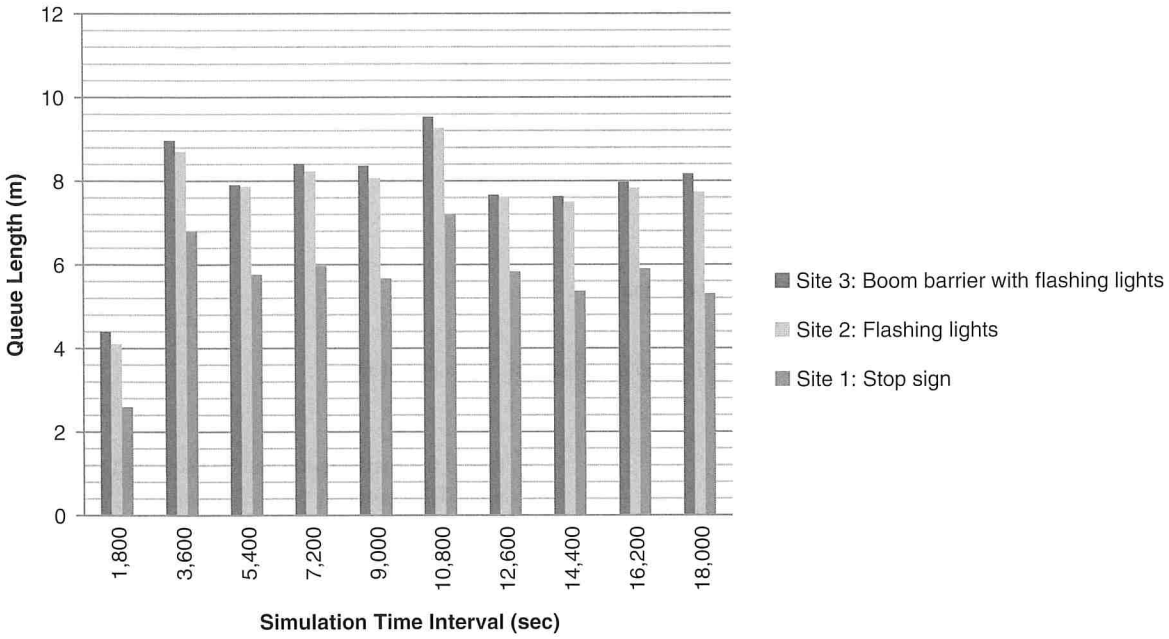
Delay and queue length are appropriate measures not only of traffic efficiency but also of traffic safety. Here, the definition of delay is the time a vehicle is stopped or its speed is reduced below a certain speed (e.g., 2 km/h) and does not exceed a certain speed (e.g., 5 km/h). When some drivers experience delays to some extent near railway crossings, they tend to change their driving behavior. They may cross the crossing illegally, make a U-turn to find an alternative route, or lose attention to proximity. Under such conditions, unsafe practices may result. As shown in Table 3, Sites 2 (flashing lights only) and 3 (boom barrier with flashing lights) show similar results on all measures. A comparison between Site 3 and Site 1 at 450 vph in each direction shows that Site 3 has 9% less total delay,

TABLE 3 Output for Delay and Queue Length

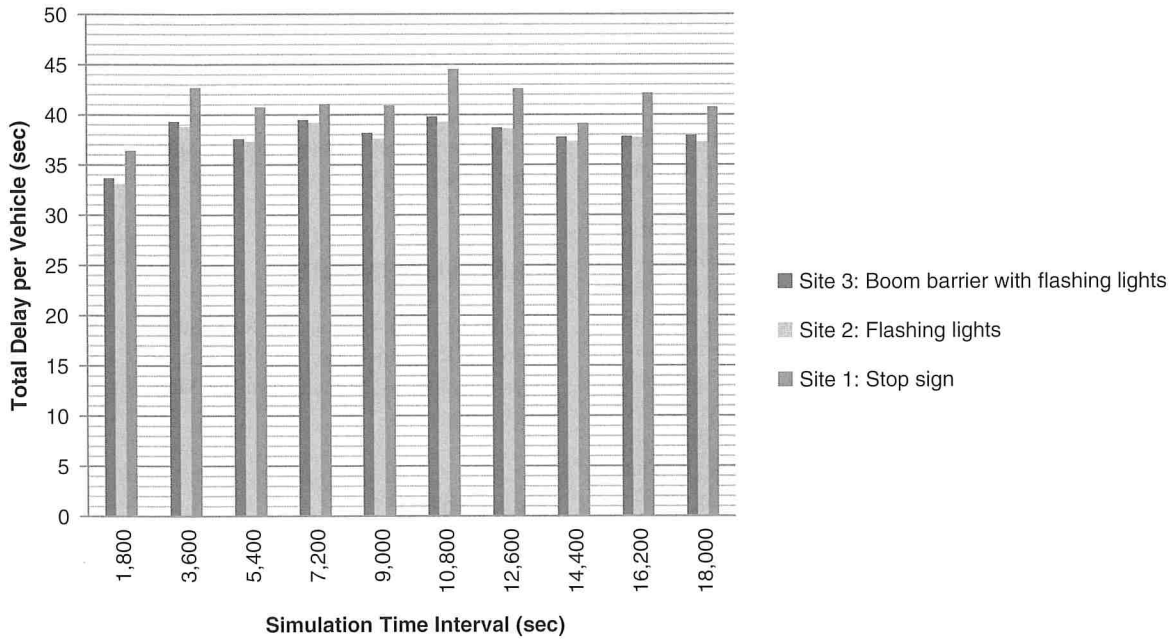
	Traffic Flow of 450 vph (each direction)			Traffic Flow of 900 vph (each direction)		
Variable	Site 3	Site 2	Site 1	Site 3	Site 2	Site 1
Total delay (h)	8.2	8.1	8.9	47.2	47.0	57.0
Total stopped delay (h)	1.2	1.2	0.8	4.0	3.9	3.1
Average queue length (m)	7.9	7.7	5.6	40.8	40.3	31.6
Number of stops	222	218	168	707	701	595
Total vehicles for 5 h	4,525	4,525	4,529	9,005	9,005	9,010

33% more total stopped delay, 29% more queue length, and 24% more vehicle stops than does Site 1. This result means that although stop signs lead to more delay, the flashing lights and the boom barrier with flashing lights caused more stopped delay, longer queue length, and more stops.

One of the advantages of using traffic microsimulation is that the results can be used to identify traffic phenomena for each specific time interval. This helps researchers focus on critical indicators. In this study, a time interval was set for every 30 min over 5 h. Figure 3*a* shows that the average queue length for Sites 2 and 3 was identical for every time interval of 450 vph in each direction. Figure 3*b* shows the average total delay that each vehicle experienced every 30 min. Each vehicle that neared the boom barrier with flashing lights and the flashing lights only experienced a delay from 33 to 40 s, but the delay was from 36 to 45 s at the stop sign. Figure 3*c* shows the way in which the number of stops changed over the 30-min period. Comparable results were obtained when the traffic flow was increased to 900 vph.



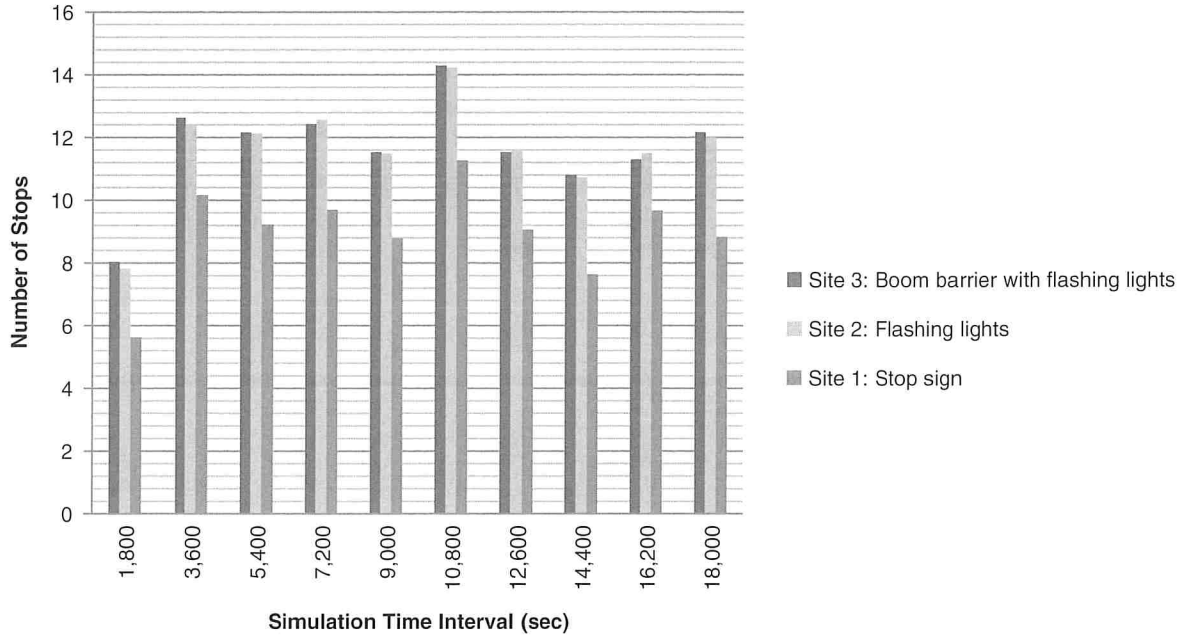
(a)



(b)

FIGURE 3 Results of 450-vph testing in each direction: (*a*) average queue length and (*b*) average total delay per vehicle.

(continued on next page)



(c)

FIGURE 3 *(continued)* Results of 450-vph testing in each direction: (*c*) number of stops.

CONCLUSIONS

Much research has been conducted on safety at road intersections. This paper shows that with the integration of surrogate safety measures and commercially available traffic simulation software, microsimulation can be used to evaluate RLXs with different types of warning devices. By incorporating actual field violation data in the model, the number of likely collisions for different warning devices was estimated. This microsimulation tool will assist in determining which safety approach is most efficient for specified traffic and train volumes. This application can be extended to assess issues of traffic queue backup over crossing from an adjacent intersection located downstream, and varying countermeasures can be evaluated. By integration with a driving simulator, this approach can be applied to evaluate and compare the safety performance of alternative warning devices such as rumble strips and in-vehicle audio warnings.

Some limitations have to be overcome in further research for a better understanding of railway safety. Occurrences of vehicle and train conflicts at RLXs are low compared with road intersection conflicts, so there is not enough data from which to draw definite conclusions. The characteristics of RLX and road intersections are different in terms of traffic volume, posted speed, and environmental setting. Another shortcoming of the study reported here is that the speed profiles were obtained from only three sites, and therefore cannot be generalized across other environments. Although other vehicle types such as trucks may possibly exhibit different vehicle and driver characteristics, traffic compositions other than passenger cars in the field were not considered in view of the nonsignificant volume. In general, it is thought that further refinements of the method are worthwhile. For improved calibration purposes, other VISSIM input parameters such as perception–reaction time, car-following characteristics, and deceleration distribution should be investigated in the field. Nevertheless, the following conclusions can be drawn from this study:

- Traffic microsimulation can be used with some modifications to evaluate RLX safety.
- The difference found in terms of a number of potential conflicts at RLXs with boom barriers with flashing lights and those with flashing lights only is relatively small, because main inputs such as speed and compliance rates measured in the field are similar for these two warning devices.
- The number of potential conflicts at stop signs is significantly higher than it is for other warning systems.
- Although delay at stop signs is higher than it is for other treatments, the total stopped delay is lower. This phenomenon implies that drivers tend to slow down but not completely stop near RLXs equipped with stop signs.
- As the stopped delay is low, the queue length is also shorter at stop signs.

ACKNOWLEDGMENTS

The authors thank Queensland Rail and the Cooperative Research Centre for Rail Innovation for their support. Li-Sian Tey acknowledges financial assistance from the Malaysia Higher Education Scholarship.

REFERENCES

1. *Highway–Rail Crossings.* Federal Railroad Administration Office of Safety Analysis. http://safetydata.fra.dot.gov/OfficeofSafety/default.aspx. Accessed April 28, 2011.
2. *Road Safety.* International Level Crossing Awareness Day (ILCAD). European Commission. http://ec.europa.eu/transport/road_safety/events-archive/2010_06_22_ilcad_en.htm. Accessed April 28, 2010.
3. *Australian Rail Safety Occurrence Data.* RR-2009-007(2). Australian Transport Safety Bureau. http://www.atsb.gov.au/publications/2009/

australian-rail-safety-occurrence-data-1-january-2001-to-31-december-2009.aspx. Accessed Jan. 13, 2010.
4. Ford, G., and A. Matthews. *Analysis of Australian Grade Crossing Accident Statistics.* Presented at 7th International Symposium on Railroad–Highway Grade Crossing Research and Safety, Melbourne, Australia, 2002.
5. Wigglesworth, E. C., and C. B. Uber. An Evaluation of the Railway Level Crossing Boom Barrier Program in Victoria, Australia. *Journal of Safety Research,* Vol. 22, No. 3, 1991, pp. 133–140.
6. Tey, L.-S., and L. Ferreira. Driver Compliance at Railway Level Crossings. In *33rd Australasian Transport Research Forum,* ACT, Canberra, Australia, 2010.
7. Tey, L.-S., L. Ferreira, and A. Wallace. Measuring Driver Responses at Railway Level Crossings. *Accident Analysis and Prevention,* Vol. 43, No. 6, 2011, pp. 2134–2141.
8. Australian Transport Council. *National Railway Level Crossing Safety Strategy.* ATC, Canberra, Australia, 2003.
9. Perkins, S. R., and J. I. Harris. *Traffic Conflict Characteristics: Accident Potential at Intersections.* Highway Research Board, Washington, D.C., 1968.
10. Garder, P. Occurrence of Evasive Manoeuvres Prior To Accidents—Data from North Carolina Accidents. *Proc., 2nd ICTCT Workshop,* Munich, Germany, 1990, pp. 29–38.
11. Glauz, W. D., and D. J. Migletz. *NCHRP Report 219: Application of Traffic Conflict Analysis at Intersections.* TRB, National Research Council, Washington, D.C., 1980.
12. Amundsen, F. H., and C. Hyden. *Proceedings of First Workshop on Traffic Conflicts.* Institute of Transport Economics, Oslo, Norway, 1977.
13. Chin, H.-C., and S.-T. Quek. Measurement of Traffic Conflicts. *Safety Science,* Vol. 26, No. 3, 1997, pp. 169–185.
14. Archer, J. Developing the Potential of Micro-Simulation Modelling for Traffic Safety Assessment. *Proc., 13th ICTCT Workshop,* Corfu, Greece, 2000, pp. 233–246.
15. Saunier, N., and T. A. Sayed. Automated Analysis of Road Safety with Video Data. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2019,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 57–64.
16. Garder, P. Pedestrian Safety at Traffic Signals: A Study Carried Out with the Help of A Traffic Conflicts Technique. *Accident Analysis and Prevention,* Vol. 21, No. 5, 1989, pp. 435–444.
17. Minderhoud, M. M., and P. H. L. Bovy. Extended Time-to-Collision Measures for Road Traffic Safety Assessment. *Accident Analysis and Prevention,* Vol. 33, No. 1, 2001, pp. 89–97.
18. Sayed, T., and S. Zein. Traffic Conflict Standards for Intersections. *Transportation Planning and Technology,* Vol. 22, No. 1999, pp. 309–323.
19. Sayed, T., G. Brown, and F. Navin. Simulation of Traffic Conflicts at Unsignalised Intersections with TSC-Sim. *Accident Analysis and Prevention,* Vol. 26, No. 5, 1994, pp. 593–607.
20. Archer, J. *Traffic Conflict Technique—Historical to Current State-of-the-Art.* Institution for Infrastructure, Royal Institute of Technology, Stockholm, Sweden, 2001.
21. May, A. D. *Traffic Flow Fundamentals.* Prentice-Hall, Englewood Cliffs, N.J., 1990.
22. Anderson, I. B., and R. A. Krammes, Speed Reduction as a Surrogate for Accident Experience at Horizontal Curves on Rural Two-Lane Highways. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1701,* TRB, National Research Council, Washington, D.C., 2000, pp. 86–94.
23. Abraham, J., T. K. Datta, and S. Datta. Driver Behavior at Rail–Highway Crossings. In *Transportation Research Record 1648,* TRB, National Research Council, Washington, D.C., 1998, pp. 28–34.
24. Lee, N. S. US-36 Environmental Impact Statement and Basic Engineering: Methodology of Grade Crossing Evaluation. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2030,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 76–84.
25. Archer, J., and I. Kosonen. The Potential of Micro-simulation Modelling in Relation to Traffic Safety Assessment. Presented at ESS Conference, Hamburg, Germany, 2000.
26. Gettman, D., and L. Head. *Surrogate Safety Measures from Traffic Simulation Models:* Final Report. FHWA-RD-03-050. Office of Safety Research and Development, Turner–Fairbank Highway Research Center, FHWA, McLean, Va., 2003.
27. Bonsall, P., R. Liu, and W. Young. Modelling Safety-Related Driving Behaviour—Impact of Parameter Values. *Transportation Research Part A: Policy and Practice,* Vol. 39, No. 5, 2005, pp. 425–444.
28. Gettman, D., and L. Head. Surrogate Safety Measures from Traffic Simulation Models. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1840,* Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 104–115.
29. Wiedemann, R., and U. Reiter. *Microscopic Traffic Simulation: The Simulation System MISSION, Background and Actual State.* CEC, Brussels, Belgium, 1992.
30. Cheu, R.-L. Calibration of FRESIM for Singapore Expressway Using Genetic Algorithm. *Journal of Transportation Engineering,* Vol. 124, No. 6, 1998, pp. 526–535.
31. Shinar, D., and S. Raz. Driver Response to Different Railroad Crossing Protection Systems. *Ergonomics,* Vol. 25, No. 9, 1982, pp. 801–808.
32. Meeker, F., D. Fox, and C. Weber. A Comparison of Driver Behavior at Railroad Grade Crossing with Two Different Protection Systems. *Accident Analysis and Prevention,* Vol. 29, No. 1, 1997, pp. 11–16.
33. Carlson, P. J., and K. Fitzpatrick. Violations at Gated Highway–Railroad Grade Crossings. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1692,* TRB, National Research Council, Washington, D.C., 1999, pp. 66–73.
34. Crooks, R., J. Cowie, N. Fletcher, R. Kempster, and I. Roulstone. *Level Crossing Equipment.* Australian Rail Track Corporation, Adelaide, South Australia, 2006.

The Highway–Rail Grade Crossings Committee peer-reviewed this paper.

Effects of Environmental Lighting Conditions on Operating Speeds on Urban Arterials

Marco Bassani and Guglielmina Mutani

Driver behavior is influenced by environmental lighting conditions on roads; in the literature, many studies report a reduced night–day accident ratio following improvements to lighting on different types of roads, with the results classified by severity and type of accident. Few studies, however, report the influence of lighting conditions on driver speed. This study investigates the principal factors that influence driver speed on arterial roads in Turin, Italy. The aim of this study was to analyze driver speed under different daylight and nighttime lighting conditions. Six arterial roads were selected for observation and the measurement of speeds and illuminance on the pavement surface. The results showed that illuminance, in addition to factors such as lane position, lane width, and the relevant speed limit, should be considered a variable that can influence driver speed. The study used a regression equation to predict operating speeds (V_{85}) on urban arterials; the corresponding sensitivity analysis has made it possible to quantify the effects of the aforementioned variables on operating speed under different environmental lighting conditions.

The design speed approach, in which the design vehicle is considered to be isolated while moving at the design speed, is still adopted for the design of rural and urban roads. The design speed is the highest one that can be selected by drivers within the constraints of the road geometry and the posted speed limits (PSLs) established for that road by the Italian National Highway Code (*1*). In this context, the designer assumes that atmospheric factors and visibility conditions do not affect the driver's field of vision. This assumption ensures that references are always to daytime conditions and to drivers whose performances are not affected by the luminance of the road environment. The only nighttime situation considered is represented by the sag vertical curve in which the radius is dimensioned on the basis of the sight distance afforded by car headlights.

The limits of this approach are evident and well documented in the literature (*2, 3*). In the advanced geometric design approach, speeds are expected to be consistent with those that drivers are likely to select. This is the reason why, nowadays, highway policies and design manuals provide tools to assist the designer with the selection of the appropriate design speed. For example, Italian standards impose the adoption of a specific driver model, the behavior of which can assist with the plotting of the design speed profile (*4*). The assessment of the speed profile is made through the adoption of the following specific safety criterion: the design speed differential between two successive horizontal elements should be below a specified threshold.

Similarly, in the United States, the Design Consistency Evaluation Module of the Interactive Highway Safety Design Model supports the designer in the assumption of appropriate design speeds through a comparison with the operating speed profile (*5*). Operating speeds are derived from experimental equations in which they are expressed as a function of the geometric characteristics of a particular road. If, in the rural environment, the main geometric variables influencing speeds are longitudinal, say, radius, curvature change ratio, and degree of curve, then in the urban environment the most important ones are transversal, that is, lane width, access density, and number of lanes in the traveled way (*6*).

Nonetheless, the speed of any individual vehicle as chosen by the driver concerned is based on many other factors such as vehicle performance and weather conditions, but most of all the speed is based on the driver's driving capability, which in turn is influenced by visibility conditions (*7, 8*). Consequently, we can state that road lighting plays a determinant role in operating speeds.

Although several studies have examined the effects of weather conditions on operating speeds, very little information is available on the effects of environmental lighting conditions and the relationship between lighting conditions and speed. The aim of this study is to partly redress this lack of knowledge in the urban environment by focusing on the derivation of operating speed equations and speed variations for different lighting conditions.

BACKGROUND AND LITERATURE REVIEW

Road Lighting Effects on Driver Behavior

In scientific literature, it is widely recognized that road lighting influences driver vision and behavior. With respect to vision, lighting systems at night increase the visibility of the roadway and roadside, thus permitting the driver to maneuver more efficiently and safely. As a result, the response time of drivers, which is the sum of their subjective perception and reaction times excluding the pedal response and the mechanical action of the brakes, is reduced.

The response time may vary from 200 ms in optimal conditions, usually associated with daytime driving (i.e., high contrast, photopic luminance), to approximately 600 ms in nonoptimal conditions experienced during nighttime driving (i.e., low luminance, low contrast,

M. Bassani, Department of Environment, Land and Infrastructures Engineering, and G. Mutani, Department of Energy, Politecnico di Torino, Corso Duca Degli Abruzzi, 24, 10129, Torino, Italy. Corresponding author: M. Bassani, marco.bassani@polito.it.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 78–87.
DOI: 10.3141/2298-09

eccentric viewing) (*9*). This modest increase in driver response time leads to a significant increase in stopping distance from 8.9 m (29.2 ft) at a speed of 80 km/h [50 miles per hour or (mph)] to 14.2 m (46.6 ft) at 130 km/h (80 mph). Plainis and Murray suggest that this increase may explain the significant increase in severity of accidents at nighttime as detailed in the following paragraphs (*9*).

In terms of behavior, road lighting leads to increased speeds and reduced levels of concentration. Assum et al. found that, on straight and curved sections of road, the increase in average speed caused by road lighting was approximately 5% and 1%, respectively (*10*). The reason for this observed increase in speed at nighttime can be partly explained by the fact that some drivers avoid driving, when possible, at nighttime; these drivers are likely to be those who would tend to drive at low speeds during daylight hours, namely elderly people and women. Consequently, young males represent a greater proportion of the total driver population during the hours of darkness. When drivers are subdivided into three age groups, as reported in Table 1, a change in the driver-age profile between daylight and nighttime becomes evident. Although women drivers represent 26% to 29% of the total during daytime, this percentage is greatly reduced during darkness hours; a similar pattern is evident for elderly people.

As luminance levels are reduced, the performance of drivers decreases (*11*). It is well established that many aspects of visual performance, such as spatial resolution, stereoscopic depth perception, and accommodation response, deteriorate under low illumination (*9*). The visibility level (VL) quantifies the ease with which the observer is able to see an object. VL is defined by the perceptibility of a small target on the road as a luminance contrast ratio according to the following formula:

$$\mathrm{VL} = \frac{\Delta L_{\text{target}}}{\Delta L_{\text{threshold}}} \tag{1}$$

where ΔL_{target} is the luminance differential between the target and its background (contrast) and $\Delta L_{\text{threshold}}$ is the luminance differential needed for minimal visibility.

VL depends on various parameters related to the object and the observer. The latter are more complicated, as they include variables such as observer visual clarity and age, the luminance level, the time of observation, and the angular dimension that the object subtends (*12*). If a 25-year-old observer has a VL of 100%, a 60-year-old person will have a VL of only approximately 57% to 58% (*13*). As a consequence, with reference to the age classes indicated in Table 1, to attain a visual performance equivalent to that of the reference group, that is, the group from 18 to 44 years of age, the group from 45 to 64 years of age needs a luminance contrast of 139%, and the group ≥65 years of age needs a luminance contrast of 220%.

TABLE 1 Driver Age Profile During Hours of Daylight and Darkness (*11*)

			Darkness		
Age (years)	Total (%)	Daylight (%)	Total (%)	Women (%)	Men (%)
18–44	62	59	67	18	49
45–64	32	33	30	7	23
≥65	6	8	3	0.5	2.5

Luminance and Illuminance

Luminance (L) is a measure of the quantity and quality of road lighting that affects driver behavior; it must be analyzed and checked with measurements taken from the driver vantage point. Conversely, illuminance (E) is the luminous flux incident on a unit area of a surface. The relationship between both variables at any point on a road surface is given by the following equation:

$$L = q \times E \cong \frac{\rho}{\pi} \times E \tag{2}$$

where q is the luminance coefficient (cd · m^{-2} · lx^{-1}) and ρ is the reflection coefficient. In Equation 2 L is in candelas per square meter and E is in lux. The second expression of L can be used only in the case of perfectly diffusing surfaces.

The luminance coefficient depends on the pavement material, the relative position of observer and luminaire, the angle of incidence of the luminous ray on the road surface, and on the angle formed by the normal to the road surface and the direction of observation (which is normally very high in magnitude). This means that every part of the road has a unique q-value (*14*). Depending on the shaded and nonshaded luminaries, the average luminance coefficient for asphalt pavements is normally a value between 0.08 and 0.14 (*15*).

The reflection properties of a road surface are given by a reduced reflection coefficient table, called the r-table, which is summarized by two metrics: diffuse reflection Q_0 and specular reflection S_1 (*16, 17*). Fotios et al. (*14*) and Frith and Jackett (*18*) found an average value of Q_0 of 0.05 in the case of asphaltic surfaces and of 0.085 for concrete-based pavement materials.

As indicated in Casol et al. (*19*), for the purposes of simplification, a road can be assumed to be perfectly diffusing with a reflection coefficient equal to πQ_0. The same hypothesis is supported by Iacomussi and Rossi (*20*), who indicate that the reflection coefficient for asphalt-based surfaces lies within the range 0.07 to 0.15. Üncü and Kayaku measured the average reflection coefficient of a road and showed this to be equal to 0.13 for an asphalt-based pavement (*12*); Fotios et al. (*14*) found an average reflection coefficient of 0.16 for asphalt-based pavement materials and 0.27 for concrete-based pavement materials.

If these findings are taken into consideration, then luminance and illuminance are two photometric variables that are influenced by pavement optical characteristics: one variable can be calculated from the other if an average luminance coefficient is assumed for the road surface from the vantage point of the driver.

Relationship Between Light, Speed, and Safety

A road's environmental lighting conditions have a significant effect on driver speed and safety. When the relationship between lighting and speed is examined, it can be observed that most of the research effort has focused on two-lane rural roads; the result is that little information is available for the urban environment.

In his pioneering work, Bennet compared the night and day speed data for two-lane rural roads (*21*). The day and night speeds were found to have a Gaussian distribution and there were limited differences between them, which allowed both sets of speed data to constitute part of the same population. In other words, it was possible to develop a single speed-prediction model that would be valid over a 24-h period. Similar observations were made by Guzman, who

found no significant differences between daytime and nighttime speeds (*22*). The magnitude of the difference was insignificant for all degrees of curvature. The same conclusions were reached by Quaium in his most recent work (*23*).

Bonneson et al. concentrated on the differences in speeds that can be observed when passenger cars are analyzed separately from trucks (*24*). In their study, the nighttime speed of the former was 3.2 km/h (2 mph) slower than the daytime speed; the nighttime speed of trucks was only 1.6 km/h (1 mph) slower.

Assum et al. measured speeds before and after the installation of road lights (*10*). By administering questionnaires and registering the lateral position of vehicles, the researchers also investigated driver concentration. For straight sections, the installation of road lighting resulted in a significant difference in speed between darkness and daylight. Speed increased by 3.6 km/h (2.2 mph) in darkness but it increased by only 1 km/h (0.6 mph) in daylight, which represents a speed increment of 3.5% in darkness over that of daylight following the installation of road lighting. The results showed that, with an increase in road lighting, speeds increased and driver concentration levels fell; moreover, the change in the driver gender and age profiles between daytime and nighttime was highlighted. This factor could explain the higher speeds observed at nighttime, as mentioned in the previous paragraph.

With respect to safety, Fotios et al. (*14*) found that the average road surface luminance (L) could be a statistically significant predictor of the effect of lighting on the night–day accident ratio (NR). The regression equation through the data is as follows:

$$\mathrm{NR} = 0.66 \times e^{-0.42 \times L} \tag{3}$$

Similarly, a study carried out on lit road sections that were at least 1 km in length and with a speed limit of 50 km/h (30 mph) found a linear relationship between the level of lighting and the NR parameter (*18*):

$$\mathrm{NR} = -0.18 \times L + 0.62 \tag{4}$$

In both cases, in the luminance range of 0.5 to 2.0 cd/m^2, lower accident rates were found on brighter sections of road. When U.S. (*25*) and European (*26, 27*) standards are referred to, the reference values for luminance are observed to be different: European recommendations encompass a wider range (0.3 to 2.0 cd/m^2) than U.S. standards (0.3 to 1.2 cd/m^2), and the average luminance for freeways is less than one-third of that for European motorways (*28*).

Comprehensive studies synthesizing the relationship between lighting and accidents have been carried out by FHWA (*29*) and by Frith and Jackett (*18*). In these studies the cost of road lighting is shown to be justified by the savings that result from a reduction in accident rates. Although estimates vary significantly, the savings can suffice to pay off a lighting installation in a few years. Observations indicate that lighting can reduce NR to within a range of values between 14% and 66%. Nevertheless, there are factors such as increased alcohol usage, fatigue, and a disproportionate presence of young drivers in nighttime traffic that affect NR values.

EXPERIMENTAL PROGRAM

The observational study described here was carried out in an urban and suburban environment only. The infrastructures considered (Figure 1) were arterial roads located in the northwest section of Italy in the city of Turin, which has a population of 1 million inhabitants within its urban confines. All these roads were built during the last great expansion of the city, which occurred between 1960 and 1970. All the maintenance work performed during the subsequent 40 in-service years has led to different transversal assets in terms of lane width and margin composition.

On this part of the urban road network, the carriageways are composed of two or three lanes with a PSL of 70 km/h (43.5 mph), with only a few sections having a limit of 50 km/h (31 mph). Only one of the main arterials of Turin, Corso Regina Margherita, was excluded from consideration for the purposes of this investigation because an automatic speed control system was installed there in 2005. The effects of that system on operating speeds were investigated by the authors in 2007. With very few exceptions, the speeds recorded were always under the speed limit of 70 km/h (43.5 mph). The conclusion reached was that the speed control system had a predominant effect on driver behavior, so this arterial was excluded from the present investigation.

Table 2 reports the general characteristics of the six arterials investigated, from which 14 road sections were selected for monitoring throughout the winter of 2009 and the summer of 2010. On all of these arterials, with the sole exception of Corso Allamano, lighting systems were present.

Section selection was dependent on identifying suitable measurement positions that were not visible to drivers and that were at a sufficient distance from the intersection to exclude conditioning effects on speeds. For most of the roads, the two sections of each carriageway were placed under observation, although in two cases, Corso Grosseto and Corso Orbassano, this was not possible. A number of the selected sections were located along tangents; the remainder were located along curves characterized by high radii greater than 280 m (918.6 ft).

Only those variables having an effect on speed were assessed or measured in the investigation:

- Traffic intensity (measured at times of free-flow conditions),
- Weather (no rainy, snowy, or foggy days, but only ones with good visibility),
- Seasonal conditions (winter and summer, which correspond to different sun heights),
- Surface pavement conditions (all measurements were taken on dry asphalt pavement),
- Road geometry (horizontal radius, lane and shoulder width, and margin characteristics),
- Vehicle characteristics and position in the carriageway (only passenger cars were evaluated), and
- Environmental lighting conditions (each piece of speed data was associated with the mean illuminance and subdivided into sunny, cloudy, or nighttime lighting conditions).

The only factors that could not be assessed were those relative to driver characteristics, such as gender, age, and driving habits (familiarity with the specific road or otherwise).

Measurement Methodology

As part of the program investigation, speed and illuminance measurements were taken under a wide variety of environmental lighting conditions during nighttime and daytime. The field investigation included the collection of speed data by means of longitudinal and

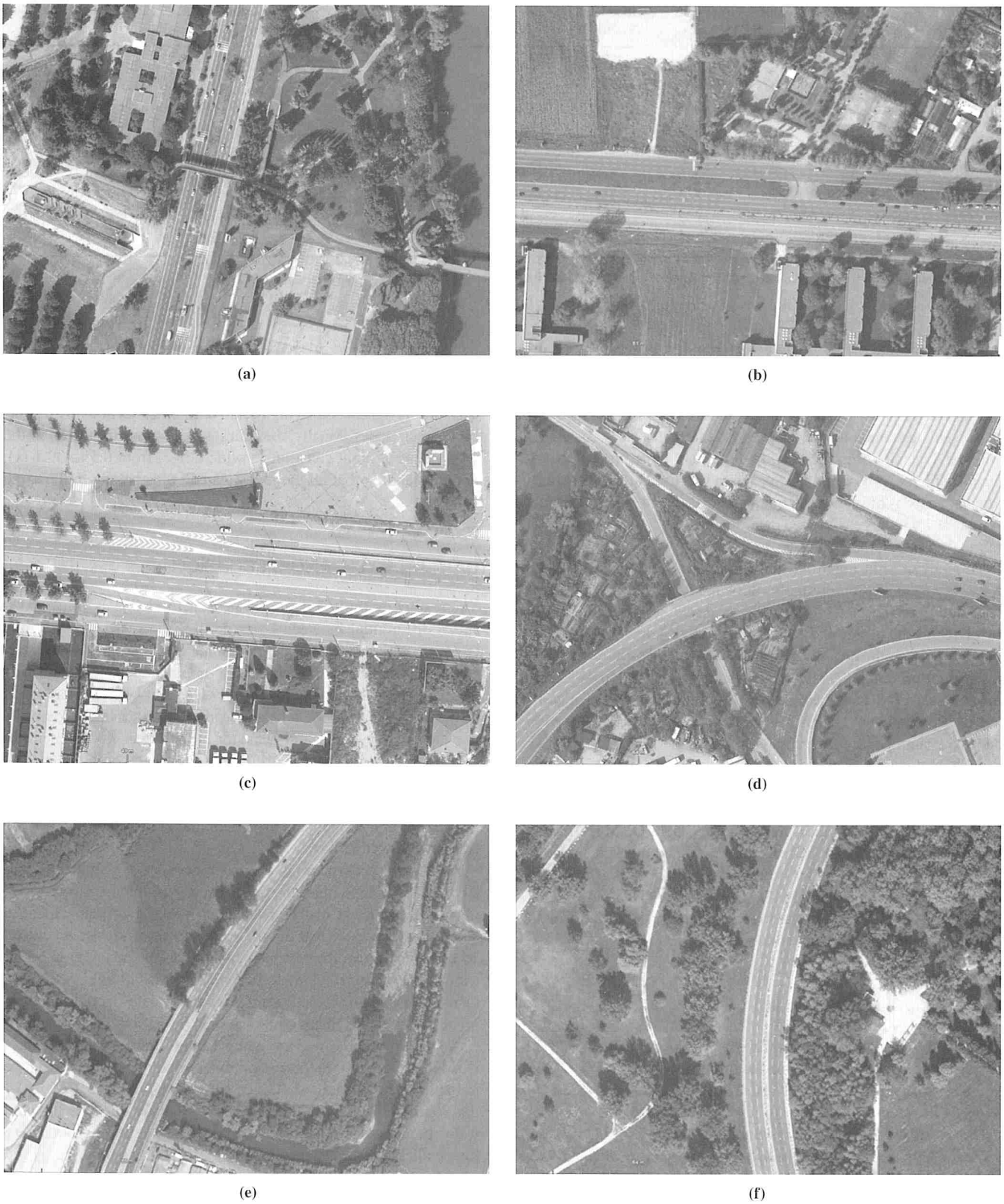


FIGURE 1 Aerial view of six arterial roads and 14 monitored sections (north is at top of figure): (*a*) Corso Unità d'Italia, (*b*) Corso Allamano, (*c*) Corso Grosseto, (*d*) Corso Orbassano, (*e*) Corso Marche, and (*f*) Lungo Stura Lazio.

TABLE 2 Sample Size and Characteristics of 14 Investigated Sections

Road Number	Road Name	No. of Sections	No. of Data	Speed Limit (km/h)	Season	Lighting Conditions	No. of Lanes	Lane Width (m)	Section Type	Radius (m)
1	Unità d'Italia	2	1,174	50–70	Winter	S, C, N	3	2.85, 2.85, 2.60	T	—
		1	1,027	70	Summer	SF, SB, C, N				
2	Allamano	1	415	50	Summer	S, C, N	3	3.60 (each)	T	—
3	Grosseto	2	648	70	Winter	S, C, N	2	3.25 (each)	T	—
		1	352	70	Summer					
4	Orbassano	2	1,316	70	Winter	S, C, N	3	3.70, 4.20, 3.70	CV	280
		1	214	70	Summer	S, C, N	2	3.40, 4.40	CV	280
5	Marche	2	652	70	Winter	S, C, N	2	3.60, 3.30	CV	920
6	Stura Lazio	2	1,546	70	Winter	S, C, N	3	3.25 (each)	CV	290

NOTE: No. = number; S = sunny; C = cloudy; N = night; SF = sun facing; SB = sun back; T = tangent; CV = curve; — = no data.

transversal measurements. Longitudinal data were collected by means of a laser speed gun. As previously mentioned, the measurement positions were located at a distance from the road and only the speed of isolated vehicles was recorded, thus avoiding the inclusion of those vehicles that could be affected by external factors associated with the dynamics of traffic flow. In the case of transversal measurements, a high-speed digital video camera was used and, from a subsequent analysis of successive frames, actual speeds were estimated. A total of 7,433 pieces of speed data were selected in more than 55 h of measurement time.

To analyze light quantity, the illuminance on a grid of points for every road section on the borders and center of driving lanes was measured. The selection of illuminance as a physical quantity is related to the easier and faster measuring methods adopted for daytime and nighttime measurement campaigns; moreover, the instrument used is less delicate. To obtain luminance from illuminance data, and because the road surfaces of the investigated arterials are all classified in Class RIII, reflection coefficients can be used to adopt the supposition of perfectly diffusing road surfaces (*16*). As a result, in this investigation illuminance was considered a surrogate variable in place of luminance, which is the main variable that affects driver visibility.

The lux meter used for measurements was a Delta OHM HD2302.0 with a 220-nm irradiance optical power meter. It measures a range of illuminance values that includes 1×10^{-2} and 1.99×10^{5} lx, with a resolution value in the range 10^{-2} to 10 lx. In every road section, illuminance was measured so as to evaluate the average value for all lighting conditions in daytime and nighttime.

Data Treatment and Results

To verify the existence of a normal distribution pattern, the speed data were subjected to appropriate statistical treatment with the chi-square and the Kolmogorov–Smirnov tests. Because the analysis yielded a positive response in all cases (a response in which at least one of the two tests was considered positive), the average speed (V_{50}), the standard deviation (SD), and the operating speed (V_{85}) were derived.

A complete summary of the three parameters is reported in Table 3, which contains the synthesis of data correlated with sunny, cloudy, and nighttime conditions, respectively. Any absence of data is the result of carriageway type (Roads 3 and 5 and Section 8 of Road 4 have two lanes only) or difficulties encountered in capturing data at night (Section 8 of Road 4). Table 3 shows that, in most cases, an increase in speed was dependent on lane position and speed limit. However, Corso Marche is an arterial with full access control that serves as a link to Turin's urban freeway (Tangenziale di Torino). This is why speeds are so high relative to other sections with similar characteristics. Finally, the illuminance values measured in the 14 sections at night constitute a wide range (from 0.5 to 39.1 lx), notwithstanding that all the investigated roads belong to the same category.

DATA ANALYSIS AND DISCUSSION

The results of Table 3 demonstrate that there is a significant V_{85} speed differential between lanes in all the investigated sections (from 62.7 to 112.7 km/h). Although the Italian National Highway Code permits the passing procedure on both sides on urban roads (*1*), most drivers still tend to consider the right lane of urban roads as the slow lane and the left lane as the fast lane. In other words, they adopt the typical rural behavior in the urban environment. In the case of rural roads, the driving rules adopted by all European countries permit passing only on the left side. In some of the analyzed cases, this speed differential is amplified by the presence of sidewalks near the right lane that lead drivers in that lane to drive with more caution at lower speeds.

A further aspect relates to the different types of shoulders, roadsides, and medians in all the considered sections. In particular, in the four sections of Corso Unità d'Italia—Road 1 or Figure 1*a*—the shoulders are nonexistent but the medians and roadsides are raised 15 cm (6 in) above the pavement. This induces the driver to adopt a more conservative behavior, particularly when illuminance is low (at night or when it is cloudy during the day), so the operating speeds in the first and third lanes are lower than those in the second lane.

A similar situation occurs along Lungo Stura Lazio, which is Road 6 or Figure 1*f*. In this case, tall longitudinal barriers are close to the lanes because the shoulders are narrow. The two sections of this road are located on the same curve but on opposite traveled ways. In particular, the third lane of Section 14 suffers from a very low available sight distance caused by limitations imposed by the presence of the barrier. In this lane the measured operating speed is 98.9 km/h (61.5 mph). This speed is lower than the operating speed of the second lane, which is equal to 103.9 km/h (64.6 mph).

The glare effect of sun position on driver behavior was analyzed in the case of Corso Unità d'Italia. During this measurement campaign, carried out in the summer of 2010 with the axis of the road aligned

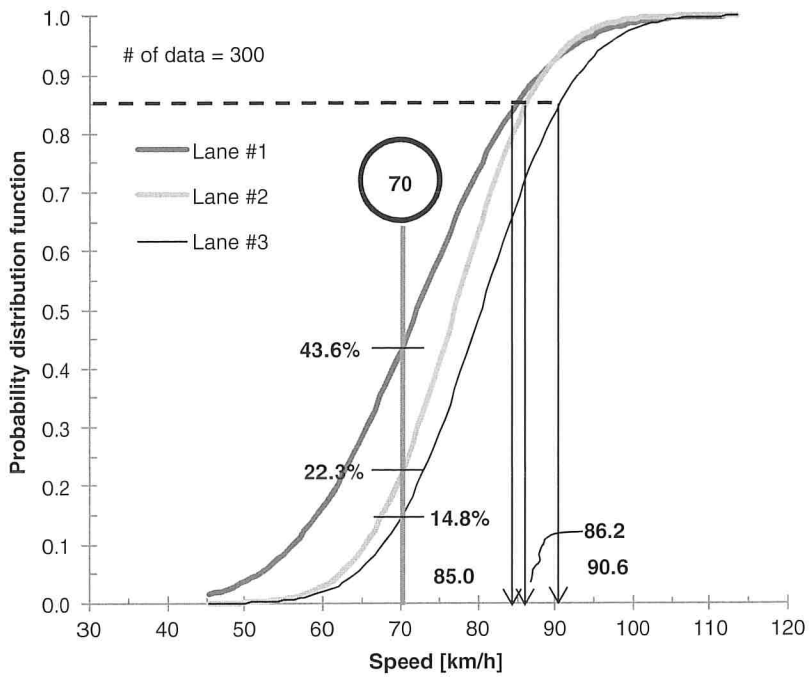
TABLE 3 Summary of Speed Results

Road Number	Section Number	PSL	E (lx)	Season	First Lane (km/h)			Second Lane (km/h)			Third Lane (km/h)		
					V_{50}	SD	V_{85}	V_{50}	SD	V_{85}	V_{50}	SD	V_{85}
Sunny Daytime													
1	1	70	77,400 (SB)	Summer	72.1	12.4	85.0	76.9	8.9	86.2	80.3	9.8	90.6
			77,400 (SF)	Summer	72.7	8.8	81.8	80.2	10.5	91.2	88.7	11.6	100.8
	2	50	23,205	Winter	59.6	8.5	68.4	67.7	9.0	77.0	68.9	11.1	80.4
	3	70	23,205	Winter	76.8	13.6	91.0	78.1	10.0	88.4	86.5	12.0	99.0
2	4	50	52,150	Summer	58.9	7.5	66.7	68.0	8.0	76.4	80.3	9.3	89.9
3	5	70	74,060	Summer	65.2	7.9	73.4	74.2	12.4	87.1	—	—	—
	6	70	22,815	Winter	76.4	11.7	88.6	88.6	12.2	101.2	—	—	—
	7	70	22,815	Winter	72.5	9.9	82.7	88.7	11.3	100.4	—	—	—
4	8	70	86,710	Summer	72.8	11.5	84.7	82.6	9.3	92.3	—	—	—
	9	70	30,420	Winter	74.3	11.0	85.8	83.2	13.7	97.5	87.2	12.5	100.2
	10	70	30,420	Winter	73.2	8.5	82.0	83.1	11.3	94.8	85.9	11.3	97.6
5	11	70	22,425	Winter	79.7	13.5	93.7	94.3	17.7	112.7	—	—	—
	12	70	22,425	Winter	79.1	14.7	94.4	89.8	10.9	101.1	—	—	—
6	13	70	21,450	Winter	69.9	7.3	77.5	71.7	6.5	78.4	70.9	9.1	80.4
	14	70	21,450	Winter	68.8	8.8	77.9	79.3	13.4	93.3	83.3	12.3	96.2
Cloudy Daytime													
1	1	70	27,530	Summer	68.8	8.7	77.8	72.6	9.6	82.6	79.9	11.5	91.8
	2	50	7,475	Winter	56.9	5.6	62.7	66.2	9.0	75.5	63.1	8.7	72.2
	3	70	7,475	Winter	72.8	6.8	79.9	74.7	5.5	80.4	79.0	5.5	84.7
2	4	50	20,710	Summer	59.5	8.8	68.6	68.3	8.7	77.3	81.9	8.7	91.0
3	5	70	43,750	Summer	73.2	11.4	85.1	82.4	12.8	95.7	—	—	—
	6	70	7,820	Winter	73.9	8.0	82.2	77.6	8.3	86.1	—	—	—
	7	70	7,820	Winter	72.7	6.3	79.3	75.8	8.6	84.8	—	—	—
4	8	70	26,600	Summer	75.0	10.0	85.4	83.7	11.3	95.5	—	—	—
	9	70	8,280	Winter	69.5	8.3	78.1	74.0	6.2	80.4	76.5	5.3	82.0
	10	70	8,280	Winter	74.6	4.7	79.5	76.2	6.4	82.8	75.0	5.6	80.8
5	11	70	6,500	Winter	77.8	8.7	86.8	77.5	9.4	87.2	—	—	—
	12	70	6,500	Winter	75.7	6.5	82.4	76.3	8.5	85.1	—	—	—
6	13	70	12,650	Winter	67.0	9.5	76.9	76.1	10.4	86.9	84.8	10.2	95.4
	14	70	12,650	Winter	69.3	8.7	78.4	78.9	11.1	90.4	85.5	10.6	96.6
Nighttime													
1	1	70	39.1	Summer	72.7	14.6	87.9	80.9	15.2	96.7	90.8	13.7	105.0
	2	50	18.6	Winter	68.3	12.2	81.0	72.9	7.7	80.9	69.2	8.3	77.8
	3	70	18.6	Winter	75.4	7.9	83.6	78.6	8.6	87.6	86.8	13.3	100.6
2	4	50	0.5	Summer	57.2	10.4	68.1	70.1	11.4	81.9	85.2	10.9	96.6
3	5	70	39.1	Summer	72.7	14.6	87.9	80.9	15.2	96.7	—	—	—
	6	70	35.8	Winter	74.0	13.0	87.5	82.3	12.0	94.8	—	—	—
	7	70	29.4	Winter	80.2	10.9	91.6	86.4	14.0	101.0	—	—	—
4	9	70	31.2	Winter	81.5	6.8	88.6	86.0	10.1	96.6	87.7	11.2	99.3
	10	70	31.2	Winter	73.0	9.6	83.0	88.5	9.8	98.7	88.7	13.0	102.1
5	11	70	23.5	Winter	83.6	9.8	93.8	86.4	13.6	100.6	—	—	—
	12	70	23.5	Winter	82.2	10.1	92.8	91.2	8.8	100.4	—	—	—
6	13	70	12.3	Winter	74.5	8.2	83.0	81.9	11.4	93.8	88.5	13.0	102.0
	14	70	12.3	Winter	82.7	12.8	96.0	92.6	10.8	103.9	88.2	10.3	98.9

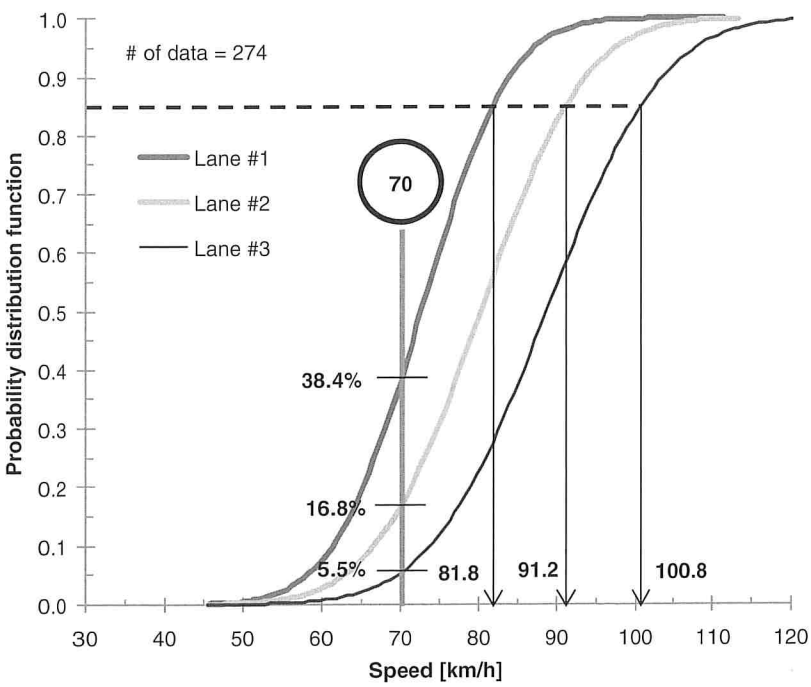
NOTE: SD = standard deviation; — = no data.

with the direction of the sun's rays, speeds were measured for both possible scenarios: with the driver facing the sun and with the driver facing away from the sun. As clearly illustrated in the two graphs of Figure 2, for the same illuminance level, the speeds in the direction facing the sun, Figure 2*a*, were lower than those recorded in the opposite direction, Figure 2*b*, because of the glare effect. Furthermore, in the sun-facing direction, the variation in operating speed (ΔV_{85}) in the three lanes was found to be lower than that for the direction facing away from the sun, passing from 5.6 km/h (3.5 mph) to 19.0 km/h (11.8 mph). Finally, the percentage of drivers respecting the PSL changed in accordance with lane position and lighting conditions. Approximately half of the speeds along the first lane of the carriageway facing the sun were lower than the PSL (70 km/h, 43.5 mph), but only 5.5% of those recorded along the third lane in the direction facing away from the sun were lower than the PSL.

The effects of environmental lighting conditions on operating speeds are represented in Figure 3, which considers the average operating speed difference (ΔV_{85}). The results seem to confirm the



(a)



(b)

FIGURE 2 Comparison for Road 1 between operating speed distribution with driver (*a*) facing sun and (*b*) facing away from sun.

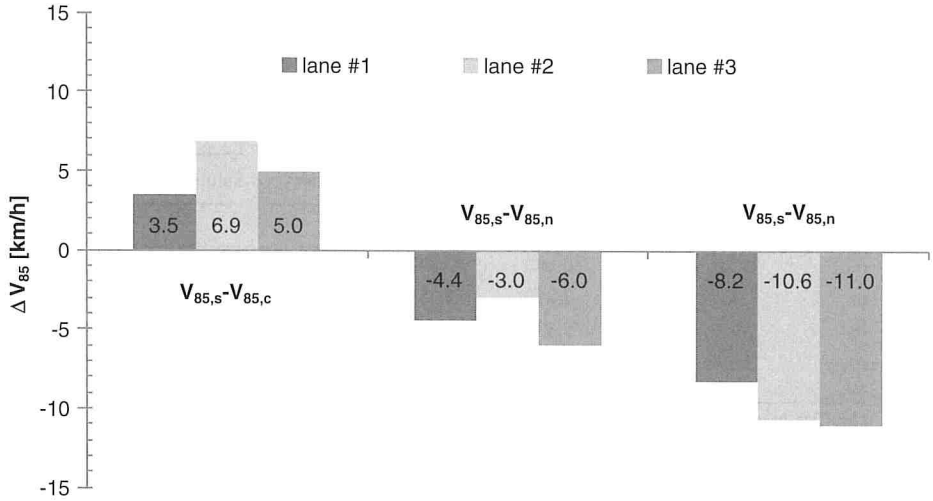


FIGURE 3 Average daytime values of operating speed variation (ΔV_{85}) for sunny ($V_{85,s}$), cloudy ($V_{85,c}$), and nighttime ($V_{85,n}$) conditions.

hypothesis that speeds are influenced not only by illuminance, as clearly indicated by the positive difference between operating speeds measured in sunny and cloudy conditions, but also by the changes in the driver population profile when daytime and nighttime data are compared. These results confirm the observation made by Assum et al. (*10*) that operating speeds at nighttime are always higher than those under any daytime lighting condition.

A regression analysis of the 85th percentile of speeds was performed that considers the most influential variables that were quantified during the field investigation. As previously indicated, illuminance was adopted in the regression analysis as a surrogate measure of luminance (L); the hypothesis of perfect diffusing pavement surfaces was assumed (*19*).

A general form of the full model was adopted for the six conditions obtained that separate the two seasonal and the three environmental lighting conditions in which a significant variation in driver population occurs. This form is necessary to take into account, although indirectly, the effects of the change in driver population age profile that occurs from daytime to nighttime and the effects of luminance variation on driver behavior. The model equation is as follows:

$$V_{85} = \alpha_0 + \alpha_1 \cdot \log_{10}(E) + \alpha_2 \cdot \mathrm{LP} + \alpha_3 \cdot \mathrm{LW} + \alpha_4 \cdot \mathrm{PSL} \quad (5)$$

where V_{85} is in kilometers per hour, E is in lux, lane position (LP) has a value in the range 1 to 3, lane width (LW) is in meters, and PSL is in kilometers per hour. Other variables such as shoulder width, margin characteristics, and curvature were found to be less significant than the selected ones.

Coefficients α_i of Equation 5 are listed in Table 4. In an attempt to find the best correlation between measured and predicted speed data, it was found that coefficient α_0 was frequently near zero. This is why coefficients α_i were recalculated, forcing α_0 to be closer to zero.

Coefficients α_i have similar values. Operating speeds depended on the illuminance on the road but were less dependent on weather and seasonal conditions. However, some anomalies in the values can be observed: α_1 for nighttime summer conditions is negative and has an opposite effect when compared with other conditions; the same observation can be made for coefficient α_3 in cloudy winter conditions, because this value is the only negative one. These anomalies merit further investigation in the future.

Table 4 also reports the coefficient of determination, which shows how much of the variation in operating speed can be explained by the four selected variables. The most significant variables are lane position and PSL; illuminance and lane width seem to be less significant.

To evaluate the weighting of variables in Equation 5, coefficients α_i were multiplied by the minimum and maximum values of the same variables. These products are reported in Table 4 as the influence range of the considered variables. From this analysis, it is possible to confirm that PSLs appreciably influenced the results for operating speeds; in fact, the product of regression coefficient α_4 and the range of speed limits (50 to 70 km/h, 31 to 43.5 mph) leads to values that vary from a minimum of 26.8 km/h (16.6 mph) to a maximum of 76.1 km/h (47.3 mph). Similarly, for the product of regression coefficient α_1 and the range of the illuminance values ($\log_{10}E$), the operating speeds vary from −16.8 to 38.9 km/h (−10.4 to 24.2 mph), with the higher gradient of influence range equal to 55.7 km/h (34.6 mph). As already reported, this result should be investigated further because it represents one of the two anomalies.

During the summer, lane position has a strong influence on operating speed and always has higher coefficients of determination with respect to other variables. During the winter, the speed limit always has higher coefficients of determination (R^2). Coefficients R^2 of the full model range from 69.6% to 95.3%.

Finally, Figure 4 shows the comparison of the 107 measured operating speeds and those derived with Equation 5. The goodness of fit is confirmed by the fact that all values are between the limits of −10% and 10%.

CONCLUSIONS

Like the geometric characteristics of road features, environmental lighting conditions exert an influence on speeds along a roadway both from a quantitative (illuminance values) and a qualitative (sunny, cloudy, and nighttime) point of view. In this study, different environmental lighting conditions for arterial roads with two and three

TABLE 4 Multiple Linear Regression Analysis and Coefficients of Determination of Selected Variables and Full Model

	Sunny Daytime		Cloudy Daytime		Nighttime	
	Summer	Winter	Summer	Winter	Summer	Winter
Coefficient of Regression Equation[a]						
α_0	−0.007	0.692	−0.042	−0.301	0.001	−0.286
α_1	6.181	3.810	2.367	7.883	−3.403	5.860
α_2	8.511	5.332	7.579	3.698	10.761	5.872
α_3	1.335	3.136	6.635	−1.414	1.494	3.781
α_4	0.536	0.786	0.604	0.725	1.088	0.921
Coefficient of Determination of Selected Variables (%)						
$\log_{10}(E)$	25.2	2.4	20.5	15.5	30.2	5.4
LP	48.7	18.7	33.5	20.3	49.0	13.6
LW	3.6	18.0	3.3	4.3	32.1	17.5
PSL	22.4	56.3	23.1	42.1	29.9	40.1
Coefficient of Determination of Full Model (%)						
R^2	80.5	80.9	84.9	69.6	95.3	72.9
Influence Range of Considered Variables (km/h)						
$\alpha_1 \cdot \log(E_{min})$	−1.9	−1.1	−0.7	−2.4	1.0	−1.8
$\alpha_1 \cdot \log(E_{max})$	30.5	18.8	11.7	38.9	−16.8	28.9
$\alpha_2 \cdot LP_{min}$	8.5	5.3	7.6	3.7	10.8	5.9
$\alpha_2 \cdot LP_{max}$	25.5	16.0	22.7	11.1	32.3	17.6
$\alpha_3 \cdot LW_{min}$	3.5	8.2	17.3	−3.7	3.9	9.8
$\alpha_3 \cdot LW_{max}$	5.9	13.8	29.2	−6.2	6.6	16.6
$\alpha_4 \cdot PSL_{min}$	26.8	39.3	30.2	36.2	54.4	46.0
$\alpha_4 \cdot PSL_{max}$	37.5	55.0	42.3	50.7	76.1	64.4

[a]Equation 5.

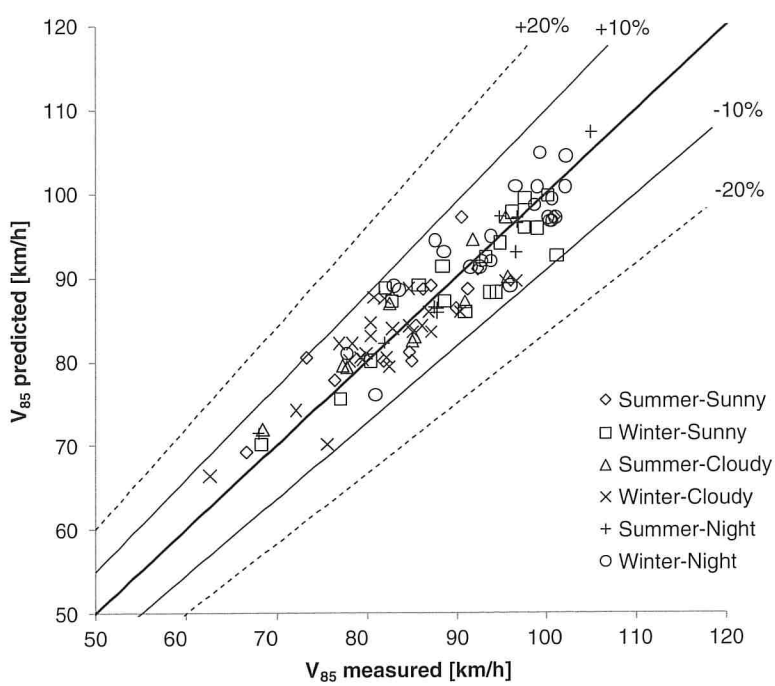


FIGURE 4 Comparison between measured and predicted values of V_{85}.

lanes and with PSLs ranging from 50 to 70 km/h (31 to 43.5 mph) were taken into account. To preclude other quantitative or qualitative factors from having an influence on driver speeds, the effects of certain curvature, weather, and visibility conditions were negated by the selection of specific road sections and days to carry out surveys.

In this observational study the main results can be synthesized as follows:

- Operating speeds (V_{85}) are influenced by the average illuminance of the road. During daytime, operating speeds increase as illuminance increases.
- V_{85} increases when drivers move from cloudy to sunny conditions. Lower speeds are observed in sunny conditions with drivers facing the sun.
- Speeds are higher at nighttime even though in darkness the illuminance values are lower than daytime. This phenomenon seems to be associated in relative terms with the increased presence of young male drivers, who tend to drive faster, as documented in literature. In the regression analysis, this phenomenon was taken into account by considering six different environmental lighting conditions, each with a different driver population profile.
- V_{85} increases as drivers move from right to left lanes. This phenomenon is also influenced by the lane width, especially at night.
- The left lane is influenced less by lighting conditions and more by the characteristics of the median and by the available sight distance. In general, drivers in the left lane go faster, independent of illuminance conditions.
- Predicted values for operating speeds seem to be in line with experimental data, so operating speeds depend on a number of variables and cannot be influenced by environmental lighting conditions alone.

The authors are aware that further studies are needed to isolate the contribution of other variables affecting operating speeds. In particular, new speed measurements should be taken that record luminance and luminance uniformity from the driver's perspective. This process will require a change in the experimental approach that will lead to an investigation into the predominant effect of age on driver visual performance. Finally, new research efforts that assess the validity of the proposed model would be valuable for road design applications.

REFERENCES

1. *Nuovo Codice della Strada* (in Italian). Decreto Legislativo 30 Aprile 1992 n. 285. Rome, Italy, 1992.
2. Lamm, R., B. Psarianos, and T. Mailaender. *Highway Design and Traffic Safety Engineering Handbook.* McGraw-Hill, New York, 1999.
3. Krammes, R. A. Design Speed and Operating Speed in Rural Highway Alignment Design. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1701,* TRB, National Research Council, Washington, D.C., 2000, pp. 68–75.
4. *Norme funzionali e geometriche per la costruzione delle strade* (in Italian). Decreto Ministeriale 5 Novembre 2001. Ministero delle Infrastrutture e dei Trasporti, Rome, Italy, 2001.
5. Interactive Highway Safety Design Model. FHWA, U.S. Department of Transportation. www.tfhrc.gov/safety/ihsdm/ihsdm.htm. Accessed July 21, 2011.
6. Wang, J., K. K. Dixon, H. Li, and M. P. Hunter. Operating Speed Model for Low-Speed Urban Tangent Streets Based on In-Vehicle Global Positioning Systems. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1961,* Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 24–33.
7. Campbell, J. L., C. M. Richard, and J. L. Graham. *NCHRP Report 600A: Human Factors Guidelines for Road Systems. Collection A: Chapters 1, 2, 3, 4, 5, 10, 11, 13, 22, 23, 26.* Transportation Research Board of the National Academies, Washington, D.C., 2008.
8. Campbell, J. L., C. M. Richard, and J. L. Graham. *NCHRP Report 600B: Human Factors Guidelines for Road Systems. Collection B: Chapters 6, 22 (Tutorial 3), 23 (Updated).* Transportation Research Board of the National Academies, Washington, D.C., 2008.
9. Plainis, S., and I. J. Murray. Reaction Times as an Index of Visual Conspicuity When Driving at Night. *Ophthalmic and Physiological Optics,* Vol. 22, No. 5, 2002, pp. 409–415.
10. Assum, T., T. Bjørnskau, S. Fosser, and F. Sagberg. Risk Compensation. The Case of Road Lighting. *Accident Analysis and Prevention,* Vol. 31, 1999, pp. 545–553.
11. Eloholma, M., J. Ketomaki, P. Orrevetelainen, and L. Halonen. Visual Performance in Night-Time Driving Conditions. *Ophthalmic and Physiological Optics,* Vol. 26, No. 3, 2006, pp. 254–263.
12. Üncü, I. S., and M. Kayaku. Analysis of Visibility Level in Road Lighting Using Image Processing Techniques. *Scientific Research and Essays,* Vol. 5, No. 18, 2010, pp. 2779–2785.
13. Adrian, W., and R. B. Gibbons. Fields of Visibility of the Nighttime Driver. *Light & Engineering,* Vol. 3, No. 3, 1995, pp. 1–12.
14. Fotios, S., P. Boyce, and C. Ellis. *The Effect of Pavement Material on Road Lighting Performance.* Sheffield Hallam University, Sheffield, South Yorkshire, U.K., 2005. http://www.ukroadsliaisongroup.org/pdfs/. Accessed July 21, 2011.
15. Forcolini, G. *Illuminazione di Esterni* (in Italian). Ulrico Hoepli Editore, Milan, Italy, 1997.
16. CIE. *Calculation and Measurement of Luminance and Illuminance in Road Lighting.* Technical Report 30-2 (TC-4-6). International Commission on Illumination, Paris, 1982.
17. CIE/PIARC. *Road Surfaces and Lighting.* Technical Report 66. International Commission on Illumination, Paris, 1984.
18. Frith, W. J., and M. J. Jackett. *Measurement of the Reflection Properties of Road Surfaces to Improve the Safety and Sustainability of Road Lighting.* Report 383. New Zealand Transport Agency, Wellington, 2009.
19. Casol, M., P. Fiorentin, and A. Scroccaro. On Road Measurements of the Luminance Coefficient of Paving. Presented at 16th IMEKO TC4 Symposium: Exploring New Frontiers of Instrumentation and Methods for Electrical and Electronic Measurements, Florence, Italy, 2008.
20. Iacomussi, P., and G. Rossi. Proprietà Ottiche e Termiche dei Manti Stradali e Condizioni di Applicazione nell'Illuminotecnica e nell'Analisi dell'Isola di Calore Urbana (in Italian). Report RdS/2010/x199. Istituto Nazionale di Ricerca Metrologica, Turin, Italy, 2010.
21. Bennet, C. R. *A Speed Prediction Model for Rural Two-Lane Highways.* Doctoral dissertation. University of Auckland, New Zealand, 1994.
22. Guzman, J. *Comparison of Day and Night Vehicular Speeds on Horizontal Curves on Rural Two-Lane Highways.* Report 04690-5. Texas Transportation Institute, College Station, 1996.
23. Quaium, R. B. A. *A Comparison of Vehicle Speed at Day and Night at Rural Horizontal Curves.* Master's thesis. Texas A&M University, College Station, 2010.
24. Bonneson, J., M. Pratt, J. Miles, and P. Carlson. *Development of Guidelines for Establishing Effective Curve Advisory Speeds.* FHWA/TX-07/0-5439-1. FHWA, U.S. Department of Transportation, 2007.
25. ANSI/IESNA. *Roadway Lighting.* Recommended Practice RP-8-00. Illuminating Engineering Society of North America, New York, 2005.
26. *Road Lighting. Selection of Lighting Class.* UNI 11248. Ente Nazionale Italiano di Unificazione, Italy, 2007.
27. *Road Lighting. Part 2: Performance Requirements. Part 3: Calculation of Performance. Part 4: Methods of Measuring Lighting Performance.* UNI EN 13201. Ente Nazionale Italiano di Unificazione, Italy, 2004.
28. Boyce, P. R. *Lighting for Driving.* CRC Press, Boca Raton, Fla., 2009.
29. *Synthesis of Human Factors Research on Older Drivers and Highway Safety.* Publication No. FHWA-RD-97-095. Research and Development, Turner–Fairbank Highway Research Center, FHWA, McLean, Va., 1997. http://ntl.bts.gov/DOCS/97095/ch02/body_ch02_04.html. Accessed April 27, 2011.

The Visibility Committee peer-reviewed this paper.

High-Beam Usage on Low-Volume Rural Roads in Texas

Vichika Iragavarapu and Kay Fitzpatrick

This study evaluated high-beam usage on low-volume roadways in Texas. Most previous studies used a subjective approach to determine high-beam usage that involved a stationary observer or an observer driving around the study section. For more efficient data collection, this study used photometric readings as the methodology. This approach allowed for a quantitative measure of high-beam use, in that it did not rely on an observer's opinion as to whether the headlamps were on high or low beam. Preliminary data collection was conducted as part of the study to estimate the threshold high-beam illuminance value; these illuminance data showed moderate use of high beams on low-volume, rural two-lane highways, with 42% of the free-flow drivers using high beams. There was a wide variation in high-beam use between sites, so site characteristics were reviewed to identify potential reasons for this variation. The data were fit to a logistic regression model to estimate the impact of various site characteristics on a driver's decision to use high beams. The results showed that vehicle speed, paved surface width, number of access points within 1,000 ft of the site, and presence of horizontal curves within 1 mi of the site were statistically significant factors that contributed to the probability of high-beam use. The results also showed that higher vehicle speeds, narrow paved surfaces, presence of horizontal curves, and few or no access points led to a higher probability of high-beam use. These results are intuitive and support the observations made in the exploratory data analysis.

One concern with nighttime driving is the ability of a vehicle's headlamps to illuminate far enough ahead of the vehicle so that the driver can safely maneuver or stop if needed, especially at high speeds. Previous studies have shown that the use of high beams can provide visibility of the downstream roadway (measured as the 3-lx line in the beam pattern on a straight road) that is 2.13 times longer than that of low beams (*1*). Data in the literature show that nighttime drivers tend to underuse their high-beam headlamps. The most relevant work on this topic was completed in Michigan, which has different terrain and traffic patterns than Texas. This paper summarizes a study conducted for the Texas Department of Transportation (DOT) on the use of high-beam headlamps on low-volume rural roadways in various regions of Texas; it also identifies potential factors that influence the choice of high-beam use. The research team collected illuminance data by mounting illuminance readers on sign faces and obtained vehicle classification and speed data by using pneumatic tubes.

Texas A&M Transportation Institute, 3135 TAMU, College Station, TX 77843-3135. Corresponding author: V. Iragavarapu, V-Iragavarapu@ttimail.tamu.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 88–95.
DOI: 10.3141/2298-10

BACKGROUND

A study in the late 1960s used a patrol vehicle to evaluate headlamp beam usage on a section of Interstate 90 in South Dakota (*2*). The driver of the patrol vehicle made complete circuits (with high beams on) around the study section and manually recorded for each vehicle encountered whether it used high beams or low beams. The study concluded that only 33% of the observed drivers were using high beams.

Another study conducted around the same time evaluated headlamp beam usage at 17 sites across the United States with varying topography and climatic conditions (*3*). Using stationary observers, researchers noted the type of vehicle, number of headlamps, direction of travel, speed, initial beam usage, beam change in response to the opposing vehicle, distance from the opposing vehicle when beam change was accomplished, presence of a leading vehicle, and trailing distances for each vehicle. The study concluded that traffic situation, area of study, and weather influenced high-beam usage and that there was an underusage of high beams (24% overall) by vehicles in situations favorable for high-beam use (i.e., rural unlighted roads with no opposing traffic and no leading vehicle within 600 ft). Researchers in that study noted that heavy trucks used high beams in 33% of the situations observed and that, in a car-meeting scenario, on average drivers dimmed their high beams at an intercar distance of 1,713 ft (522 m).

A more recent study completed in Michigan also involved stationary observations of headlamp use (*4*). The study was conducted on three different roadways free of fixed illumination in rural areas. Roadside photometric measurements at a distance 328 ft (100 m) forward of the approaching so-called clear vehicle, at a height of 5 ft (1.52 m) and at 24.6 ft (7.5 m) to the left side of the roadway, were taken for one direction of traffic to support the observer judgment. Clear vehicles were defined as those that were unopposed by any approaching traffic in the opposite lane within visible distance, were not following a leading vehicle, and were not followed by another vehicle. The study determined high-beam usage to be around 50% on the basis of observed judgment and 42% on the basis of illuminance readings.

Another such study, also conducted in Michigan, investigated high-beam use from field operational test data for two crash warning systems (*5*). The data acquisition system in the instrumented vehicles used in the crash warning systems study provided information on headlamp state (off, low, or high) and location (rural or urban) that was based on Global Positioning System information. The study concluded that under most favorable conditions (i.e., rural roads, no opposing traffic, and no leading or following traffic), high-beam usage was about 25%. Furthermore, drivers used high beams more frequently on local roads than they did on limited access roads.

Because driver age and gender were known, the study compared high-beam usage with driver age and gender. The conclusion was made that older drivers used high beams more often than younger drivers. The effect of gender was not statistically significant.

A study comparing U.S. and European high-beam maximum intensities noted, on the basis of the sparse data available, that European high-beam usage was much higher than that in the United States (*6*). The author of the study speculated that one of the reasons for this higher usage might be the considerably lower intensity of high beams in the United States (75,000 cd, compared with 140,000 cd in Europe).

STUDY OBJECTIVE

The objective of this Texas DOT study was to evaluate high-beam usage on low- to medium-volume roadways across Texas. The site characteristics were also reviewed to identify potential factors that influence the choice of high-beam use.

DATA COLLECTION

For more efficient data collection, this study used photometric readings as the methodology. This approach allowed a quantitative measure of high-beam use; whether or not the head lamps are on high beam is not a matter of opinion.

A Minolta T-10M illuminance meter unit was used in this study. The unit has small waterproof receptors that can be easily controlled with a portable computer, and it has an extremely large measuring range of 0.01 to 299,000 lx.

Preliminary Data Collection

To evaluate the illuminance meter for data collection purposes and to estimate the threshold illuminance value of a standard headlamp on high beam, the researchers conducted preliminary data collection on a two-lane rural road in Bryan, Texas. Eight passenger vehicles with headlamp heights from 27 in. to 36 in. were used for this purpose. These headlamp heights are within the range of average headlamp height for passenger cars as given in Table 31 of NCHRP Report 400 (*7*).

The illuminance readings were taken for one direction (westbound) of traffic with the receptor mounted on a roadside sign. The receptor was placed on the bottom of the sign, at 13.8 ft lateral and 7 ft vertical distance from the right edge line of the lane. Pneumatic tubes were installed at the sign location to confirm vehicle classification and speed. Figure 1 shows the equipment setup for preliminary data collection.

Each vehicle was driven through the setup with low beams and high beams, under free-flow conditions (i.e., when there was no following, leading, or opposing traffic). The drivers used their cruise control on the approach to the sign so that the vehicle would be at a constant speed throughout the setup. The time when the volunteer vehicle crossed the tubes for each beam condition was noted and was used to match the tube data with the illuminance meter readings. The peaks observed in the illuminance data were used to identify the presence of a vehicle; these data were then matched with the tube data to obtain vehicle class and speed. The speed of the vehicle measured at the sign was used to convert the illuminance level by time readings to illuminance level by distance upstream of the sign.

FIGURE 1 Equipment setup for preliminary data collection.

Figure 2 shows the peak illuminance readings from each test vehicle on low and high beams. The graph shows that the peaks for high-beam light levels were between slightly less than 3.0 lx and 7.3 lx. The peaks for low-beam light levels were below 1 lx. For this research effort, a value of 2.5 lx was selected as the illuminance threshold for classifying a vehicle's beam usage.

Field Data Collection

The same setup used for preliminary data collection was used for field data collection. One illuminance meter receptor was placed on the face of a traffic sign. The sensor was located approximately 7 ft above the road surface, which is the height that signs are to be placed according to Figure 4-15 in the Texas DOT *Sign Crew Field Book* (*8*). The lateral offset from the edge line was between 13.0 and 16.3 ft. Pneumatic tubes were located adjacent to the sign with the illuminance meters. The data recorder was chained to the base of a traffic sign pole to secure it from vandalism.

Site Selection

The research team did not select higher-volume roadways for illuminance data collection because drivers are less likely to use high beams under such conditions. All sites for illuminance data collection were two-way, two-lane undivided highways with no roadway lighting. Farm-to-market roadways were typically selected; however, for sites in west Texas, U.S. and state highways were selected. For more efficient data collection, research team members sought data collection locations with two signs that were close to one another but facing opposite directions. The sites were all located on a flat, straight segment of the roadway. Upon arrival at the data collection location, the data collection team confirmed that the speed limit was posted properly at both ends of the roadway segment, to avoid speed changes on the approach. Narrow roadways with very little clear zone and no room to maneuver the equipment truck were avoided. Additionally, the data collection team sought sign faces

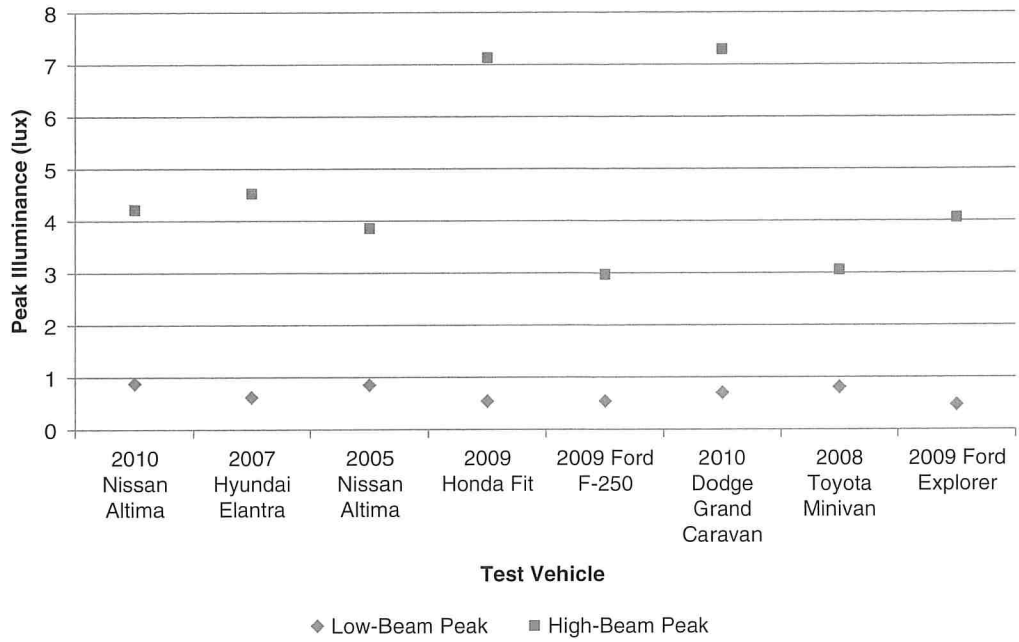


FIGURE 2 High-beam illuminance readings from preliminary data collection.

with horizontal curvature more than 500 to 1,000 ft from the data collection point. Data were collected between June and November 2010 at nine sites (18 approaches).

DATA REDUCTION

The processed illuminance data contain peaks, which represent the presence of a vehicle. The time stamps present in both data sets were used to match these peaks with the tube data. For each vehicle, the illuminance data provided the peak illuminance value and the matched tube data set provided the vehicle class and speed. The following assumptions were made in data reduction:

- The time from 9 p.m. to 5 a.m. represented dark conditions at these sites, and data collected between these times were retained in the database.
- A minimum illuminance value of 0.1 lx was used to clean the data.
- A minimum vehicular speed of 40 miles per hour (mph) was used to further clean the data.
- High-beam usage was considered to be greater than 2.5 lx (on the basis of preliminary data collection results).
- The following vehicle classification was used:
  - Car = tube class of 1, 2, 3, 5, and 6;
  - Truck = tube class of 4, 8, 9, 10, and 13; and
  - Unknown = tube class of 11, 14, or any other.
- Only cars were retained in the database.
- Only clear vehicles were retained in the database. Clear vehicles were defined as
  - Vehicles under free-flow condition, that is, a headway of more than 5 s and tailway of more than 3 s, and
  - Vehicles with no opposing traffic, that is, no vehicle in the opposite lane within 1,700 ft [on the basis of the value observed in previous research (*3*)].

RESULTS

Overall High-Beam Usage

A total of 1,869 clear cars (free flowing with no opposing vehicle) were included in the analysis. Figure 3 shows the cumulative distribution of peak illuminance values for these 1,869 cars. The curve represents the percentage of cars with a given illuminance value or less. To determine the percentage of high-beam use, subtract the curve value from 1. For example, at illuminance level 2.5, 42% of drivers are using high beams (1.00 – 0.58). Table 1 summarizes high-beam usage and site characteristics for each site for the data collection period, by approach. The table also shows that approximately 42% of the cars were using high beams.

The results in Table 1 show that high-beam usage varied across the sites, from as low as 1% at Site 10 (Approach 1) to as high as 78%

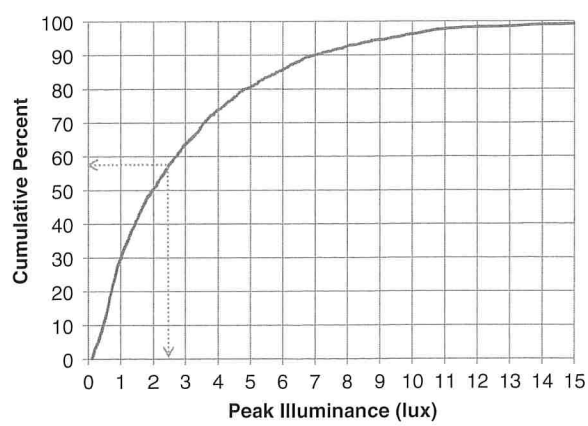


FIGURE 3 Cumulative percentage of peak illuminance reading for 1,869 clear cars.

TABLE 1 High-Beam Usage and Site Characteristics by Approach

Site_ID (Approach)	Number of Cars	High-Beam Use (%)	Posted Speed Limit (mph)	Width (ft)		Horizontal Curves Within 1 mi?	Access Points (1,000 ft either side)
				Lane	Shoulder		
1 (1)	92	27	65	12	4	No	1
1 (2)	108	36	65	12	4	No	1
5 (1)	152	34	65	10	0	Yes	9
5 (2)	84	18	65	10	0	Yes	9
6 (1)	111	64	65	10	0	Yes	1
6 (2)	81	42	65	10	0	Yes	1
7 (1)	116	21	65	12	4	No	0
7 (2)	90	50	65	12	4	No	0
8 (1)	142	32	65	12	8	No	1
8 (2)	69	26	65	12	8	No	1
9 (1)	208	78	65	11	0	Yes	0
9 (2)	196	73	60	11	0	Yes	0
10 (1)	113	1	65	12	8	No	0
10 (2)	97	14	65	12	8	No	0
11 (1)	59	56	65	11	0	Yes	5
11 (2)	76	57	65	11	0	Yes	5
12 (1)	41	27	65	12	0	Yes	0
12 (2)	34	47	60	12	0	Yes	0
Total	1,869	42	—	—	—	—	—

NOTE: — = not applicable.

at Site 9 (Approach 1). This variance implies that site characteristics probably influence a driver's choice to use or not to use high beams.

Comparison of Texas High-Beam Usage with Literature

In their 1968 study, Hare and Hemion found approximately 24% of the vehicles used high beams under clear conditions (*3*). However, in their 2003 study, Sullivan et al. reported a high-beam usage of 42% with their illuminance criterion of 1.05 lx and a high-beam usage of 50% from their observer judgment (*4*). Figure 4 shows a comparison of per hour results obtained from this Texas study with those of other studies on high-beam usage (*3, 4*). Beam usage for this study is not shown for fewer than 5 vehicles per hour (vph). The figure shows that the volumes at the sites used in this Texas study were generally lower than those of the other studies. For the 279 hours of data available at the Texas sites, the maximum clear

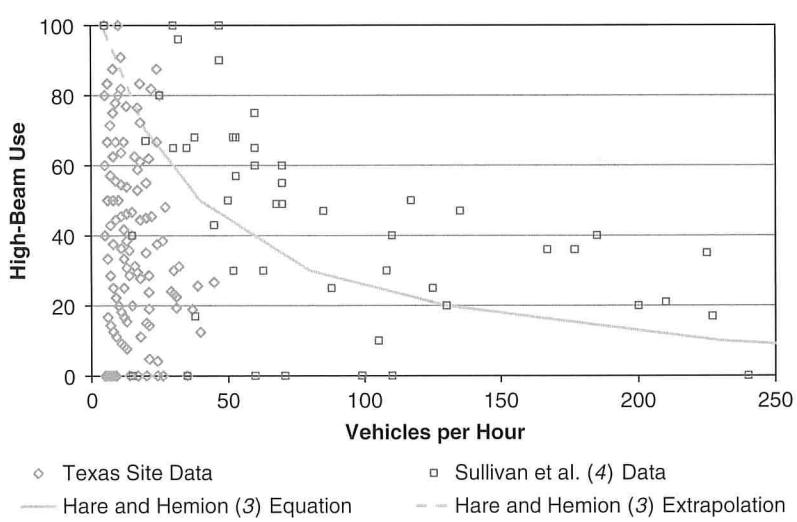


FIGURE 4 Comparison of results for Texas data and previous studies.

volume was only 45 vph. Sullivan et al. had data for a volume as high as 236 vph (*4*). Each study shows an overall trend of decreasing high-beam usage with increasing traffic volume; however, there is a lot of variability in high-beam use at the lower volumes. Overall, at the low-volume conditions of the Texas study sites it was observed that the high-beam usage was moderate, given the assumption of 2.5 lx as the threshold illuminance value.

Influence of Site Characteristics on High-Beam Usage

Exploratory Analysis

The characteristics of the sites were reviewed to identify potential reasons for the variation in high-beam use. Table 2 summarizes the site characteristics used in the analysis for each site, by approach.

Figure 5 shows high-beam use by the paved surface width, namely 2 × (lane width + shoulder width). For the two-lane highways with a larger number of narrow surface widths (i.e., more narrow lanes or shoulders), the high-beam use was higher in many cases.

Another characteristic checked was whether a site being within 1 mi of a horizontal curve resulted in greater high-beam use. Four of the sites (i.e., eight approaches) were not within 1 mi of a horizontal curve. Those sites had an average of 26% high-beam use. The remaining sites were within 1 mi of a horizontal curve and had an average of 49% high-beam use. Figure 6 illustrates the comparison. The presence of a horizontal curve appeared to influence a driver's decision on whether to use high beams.

A similar comparison was done with the number of access points (see Figure 7). For the two sites (i.e., four approaches) with more than one access point within 1,000 ft of the site, the average high-beam use was 41%. The remaining sites had one access point or fewer within 1,000 ft of the data collection location and had a large range of high-beam use—between 1% and 78%, with an average of 38%. The influence of the presence of access points on a driver's decision to use high beams is not clear.

The pneumatic tubes provided the speed of each vehicle. The speed for cars using high beams was compared with the speed for cars not using high beams. Figure 8 illustrates the average speed for high-beam users and low-beam users for each site. For most of the sites, the average speed for drivers using high beams was slightly higher than the average speed for drivers using low beams. Although at a few sites the difference in speed between drivers using high beams compared with those using low beams appears to be noteworthy, overall the speeds were similar.

Logistic Regression Model

The data were evaluated to identify potential factors that influenced the choice of high-beam use. Given the categorical nature of the choice of high-beam use (i.e., yes or no), a logistic regression model was used to describe the relationship between the choice of high-beam use and site characteristics (*9*). A logistic regression model to describe the relationship between high-beam usage and various site characteristics can be expressed as

$$\begin{aligned}\text{logit}(\pi) &= \log\left[\frac{\pi}{1-\pi}\right] = \alpha + \beta_1 * x_1 + \beta_2 * x_2 + \cdots + \beta_k * x_k \\ &= \alpha + x\beta \end{aligned} \tag{1}$$

TABLE 2 Site Characteristics by Approach in High-Beam Usage Analysis

Site_ID (Approach)	Posted Speed Limit (mph)	Paved Surface Width (ft)	Horizontal Curves Within 1 mi?	Access Points (1,000 ft either side)[a]	Average Speed of Car (mph)	
					High Beam	Low Beam
1 (1)	65	32	No	No	68.8	66.6
1 (2)	65	32	No	No	68.8	62.7
5 (1)	65	20	Yes	Yes	61.3	59.0
5 (2)	65	20	Yes	Yes	59.4	55.7
6 (1)	65	20	Yes	No	64.4	64.8
6 (2)	65	20	Yes	No	64.5	63.0
7 (1)	65	32	No	No	69.0	69.5
7 (2)	65	32	No	No	70.8	67.8
8 (1)	65	40	No	No	64.5	61.3
8 (2)	65	40	No	No	61.8	64.4
9 (1)	65	22	Yes	No	63.1	59.5
9 (2)	60	22	Yes	No	62.6	59.4
10 (1)	65	40	No	No	63.0	67.9
10 (2)	65	40	No	No	68.3	64.9
11 (1)	65	22	Yes	Yes	63.0	65.3
11 (2)	65	22	Yes	Yes	67.2	65.6
12 (1)	65	24	Yes	No	69.4	66.3
12 (2)	60	24	Yes	No	66.3	63.3

[a]Yes = >1; no = ≤1.

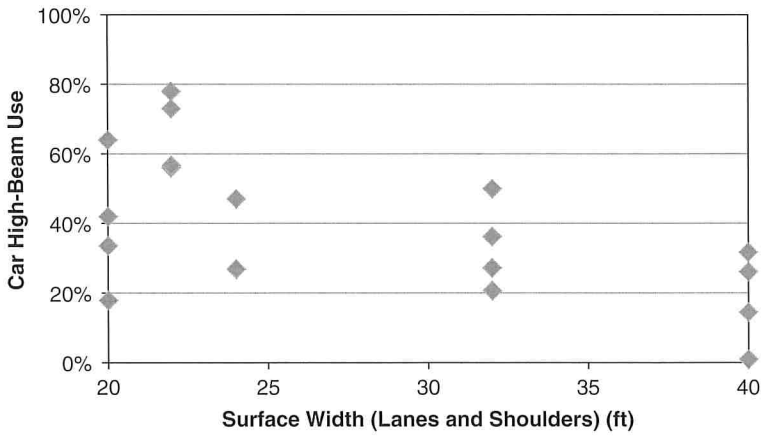


FIGURE 5 Car high-beam usage by paved surface width.

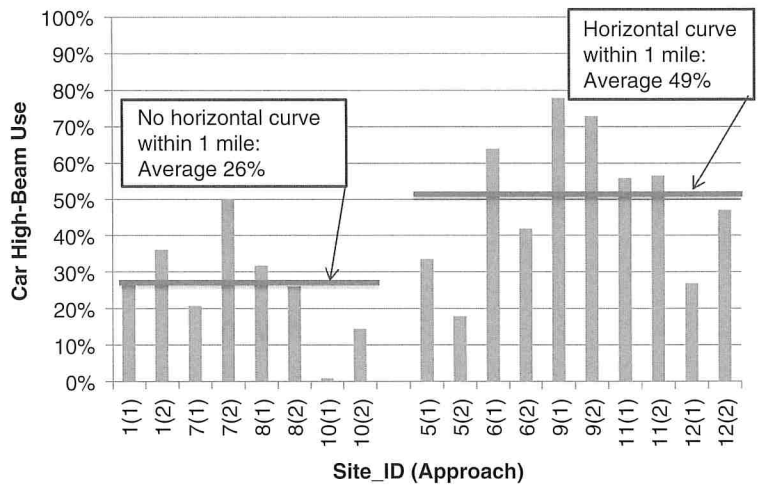


FIGURE 6 High-beam usage by presence of horizontal curve.

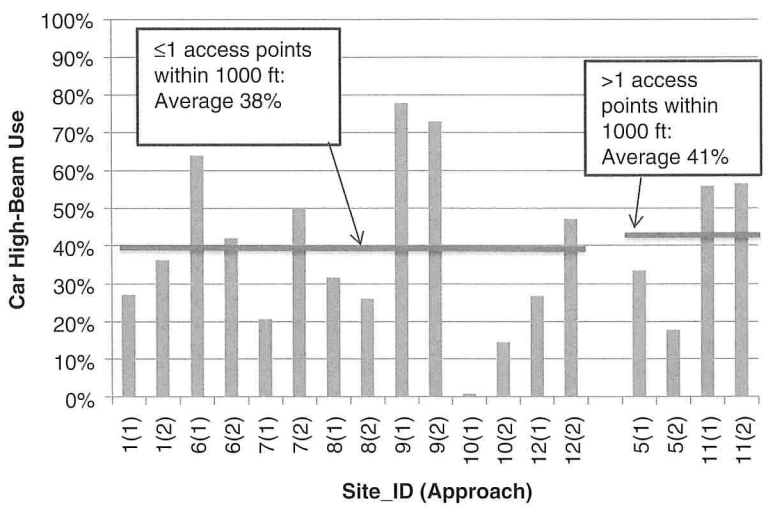


FIGURE 7 High-beam usage by number of access points within 1,000 ft.

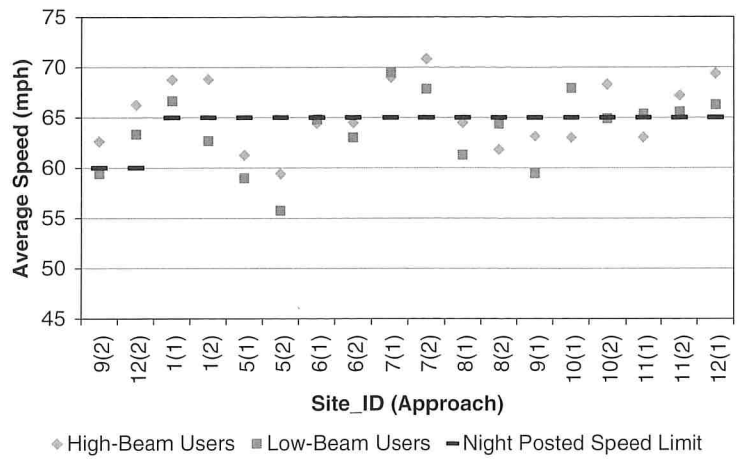


FIGURE 8 Average speed of high-beam and low-beam car drivers.

where

π = probability of high-beam usage,
α = intercept term,
β = estimated coefficient for each site or vehicle characteristic,
x = site or vehicle characteristic, and
k = number of site or vehicle characteristics considered in the model.

Equation 1 uses the logit scale, which can be converted to the log of odds back to the probability such that

$$\pi = \frac{\exp(\alpha + x\beta)}{[1 + \exp(\alpha + x\beta)]} \tag{2}$$

Table 3 summarizes the model estimates for the probability of use of high beams. The results show that vehicle speed, paved surface width, access points within 1,000 ft of the site (≤1 or >1), and presence or absence of horizontal curves within 1 mi of the site were statistically significant factors that contributed to the probability of high-beam use. Hourly volume was not significant; the volumes were very low for the study sites, especially when compared with those of the other studies in the literature. The results show that higher vehicle speeds, narrow paved surface widths, presence of horizontal curves, and few or no access points were associated with a higher probability of high-beam usage. These results are intuitive and support the observations made in the exploratory data analysis.

TABLE 3 Model Estimates for Probability of High-Beam Usage

Parameter	df	Estimate	Standard Error	*p*-Value
Intercept	1	−0.7523	0.5876	.2004
Vehicle_Speed	**1**	**0.0278**	**0.0061**	**<.0001**
Hourly_Volume	1	−0.0163	0.0113	.1508
Hour 0	1	0.0678	0.2220	.7600
Hour 1	1	0.0631	0.2574	.8063
Hour 2	1	−0.0136	0.2733	.9063
Hour 3	1	−0.2667	0.2798	.3405
Hour 4	1	0.0903	0.2136	.6725
Hour 21	1	−0.1364	0.1727	.4294
Hour 22	1	−0.0220	0.1743	.8995
Paved_Surface_Width	**1**	**−0.0578**	**0.0196**	**.0032**
Access_Pts_1000 = 0	**1**	**1.0179**	**0.1448**	**<.0001**
HC_1mi = 0	**1**	**−0.9678**	**0.2981**	**.0012**

NOTE: Bold type indicates statistically significant variable. Access_Pts_1000 = access points within 1,000 ft of site (≤1 or >1); HC_1mi = presence or absence of horizontal curves within 1 mi of site. Model fit statistics as follows:

Criterion	*Intercept only*	*Intercept and covariates*
Akaike information criterion	2,576.322	2,313.734
Schwarz criterion	2,581.865	2,385.804
−2 log likelihood	2,574.322	2,287.734

SUMMARY

The Texas illuminance data show a moderate use of high beams on low-volume, rural two-lane highways, with 42% of clear cars using high beams. This value is similar to that observed by Sullivan et al. in their 2003 study, namely 50% based on observed judgment and 42% based on illuminance readings (*4*), and it is higher than that reported by Hare and Hemion in their 1968 study, namely 24% (*3*). The higher Texas high-beam usage could be due to the lower volumes at the sites included in this study. The Texas sites in this study had a maximum of 45 vehicles per hour during the nighttime hours as compared with 500 vehicles per hour reported by Hare and Hemion and 236 vehicles per hour reported by Sullivan et al. (*3, 4*).

The range of high-beam use was between 1% and 78% at the Texas study sites. This range indicates that site characteristics influence a driver's decision on whether or not to use high beams. Potential influences identified in this study include vehicle speed, paved surface width, number of access points within 1,000 ft of the site, and presence or absence of horizontal curves within 1 mi of the site. Higher vehicle speeds, narrow paved surface, few or no access points, and presence of horizontal curves were associated with a higher probability of high-beam use.

CONCLUSIONS

The conclusions from this study are as follows:

- Overall high-beam use for this sample of 1,869 clear cars on low-volume, rural two-lane Texas highways was 42%. The

range of high-beam use was between 1% and 78% across the study sites.

- Site characteristics influence the decision of high-beam use. Potential factors include
  - Vehicle speed,
  - Paved surface width,
  - Number of access points within 1,000 ft of the site, and
  - Presence or absence of horizontal curves within 1 mi of the site.
- High-beam use is more likely at higher vehicle speeds.
- Higher high-beam use was observed on roadways with a larger number of narrow surface widths (lane width plus shoulder width).
- Having few or no access points within 1,000 feet of the study site is associated with higher high-beam use.
- The presence of a horizontal curve within 1 mi of the study site is associated with higher high-beam use.
- A trend of higher high-beam use for lower traffic volumes is observed in the literature.

ACKNOWLEDGMENTS

This paper is based on research sponsored by FHWA and the Texas DOT. The project is under the direction of Darren McDaniel of the Texas DOT. The research was performed at the Texas A&M Transportation Institute (TTI). The authors thank James Robertson and Travis Gilbert, students at Texas A&M University, for their help with data collection. Thanks also to Paul Carlson and Jeff Miles of TTI for their valuable input on study methodology and data analysis.

REFERENCES

1. Sivak M., B. Schoettle, T. Minoda, and M. J. Flannagan. *Potential Visibility Gains on Straight and Curved Roads from Proportional Increases in Current Low-Beam Intensities.* Report UMTRI-2005-34. University of Michigan Transportation Research Institute, Ann Arbor, 2005.
2. Cheeseman, M. R., and W. T. Voss. Motor Vehicle Headlight Beam Usage on a Section of Interstate Highway 90. In *Highway Research Record 164,* HRB, National Research Council, Washington, D.C., 1967.
3. Hare, C. T., and R. H. Hemion. *Headlamp Beam Usage on US Highways.* Final report on Phase III. AR-66. Southwest Research Institute, San Antonio, Tex., 1968.
4. Sullivan, J. M., G. Adachi, M. L. Mefford, and M. J. Flannagan. *High Beam Headlamp Usage on Unlighted Rural Roadways.* Report UMTRI-2003-2. University of Michigan Transportation Research Institute, Ann Arbor, 2003.
5. Mefford, M. L., M. J., Flannagan, and S. E. Bogard. *Real World Use of High Beam Headlamps.* Report UMTRI-2006-11. University of Michigan Transportation Research Institute, Ann Arbor, 2006.
6. Rumar, K. *Relative Merits of the U.S. and ECE High Beam Maximum Intensities and of Two- and Four-Headlamp Systems.* Report UMTRI-2000-41. University of Michigan Transportation Research Institute, Ann Arbor, 2000.
7. Fambro, D. B., K. Fitzpatrick, and R. J. Koppa. *NCHRP Report 400: Determination of Stopping Sight Distances.* TRB, National Research Council, Washington, D.C., 1997.
8. *Sign Crew Field Book.* Texas Department of Transportation, Austin, 2009. http://onlinemanuals.txdot.gov/txdotmanuals/sfb/sfb.pdf. Accessed April 28, 2011.
9. UCLA Technology Services. Statistical Computing Seminar. http://www.ats.ucla.edu/stat/sas/seminars/sas_logistic/logistic1.htm Accessed on July 18, 2011.

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or polices of the Texas DOT.

The Visibility Committee peer-reviewed this paper.

Development of Visual Model for Exploring Relationship Between Nighttime Driving Behavior and Roadway Visibility Features

Ronald B. Gibbons, Christopher J. Edwards, Rajaram Bhagavathula, Paul Carlson, and D. Alfred Owens

Driving is often considered a visually oriented task. This visual task is constrained when drivers drive at night. Visibility is reduced because visual cues available during the day are not present at night. This study attempted to develop a link between driver safety and the nighttime visual environment. This research required creating and integrating new technology, observing and collecting data, and developing a model framework called the Dynamic Driver Visual Model (DDVM). Conceptually, a DDVM is a system of rules, statistics, and expectations that can be used to define how a driver collects visual information from the environment. Several information sources were investigated, and several dependent variables were identified. Data were collected on how information from signage, objects, lighting, pavement markings, and other vehicles moderates a driver's visual search of the roadway environment. Several logistic regression analyses were performed on the collected data to identify common characteristics to be implemented in the DDVM. These variables included age, lighting, vehicle headlamps, several different objects, glance time, target luminance, and contrast information. The results suggest that a number of target and visibility elements have nonlinear effects on a driver's detection performance at a variety of detection distances. This paper discusses the implications of these findings and the initial framework of the DDVM. Future research and additional data requirements are also discussed.

Visibility models are based on the fact that visibility levels of targets at the time of detection depend on a number of factors. Age, headlamp beam pattern, target reflectivity, and contrast have all been shown to have a significant effect on visibility level at target detection (*1, 2*). Visual complexity, particularly of the area immediately around a target, should also be taken into account. Targets set against highly complex backgrounds, such as pedestrians in urban environments, have a lower probability of detection and require higher visibility levels (*3*). Conversely, less complex backgrounds do not require such high visibility for an acceptable probability of target detection.

The most commonly cited visibility model was developed by Adrian (*4*). According to Adrian, the visibility level of the target is defined as the luminance difference between the target and its background in actual road conditions (ΔL actual) and the luminance difference needed for minimal visibility (ΔL threshold). Rea and Ouellette outlined a quantitative visual model that related measurable environmental aspects to measurable human responses and was independent of nonvisual factors (*5*). This model, termed relative visual performance (RVP), is based on the object's size and the retinal illuminance of the object, among other factors.

There are several aspects of the real world that do not fit into the simplified RVP model. Some of these aspects include luminance, geometry, visual complexity, motion, and driver attention. The study reported here attempts to incorporate these aspects of a visual model into a simplified statistical methodology.

MODEL FRAMEWORK

The development of the dynamic driver visual model (DDVM) required consideration of the statistical visibility of an object in the roadway. This framework needed to incorporate factors that directly correlate with and aid in nighttime visibility. Elements such as age, luminance, glance duration, object type, free glance, detection, and object recognition were initially identified as influencing driver behavior at night.

The initial model framework was the probability of object detection and visual distance functions for each object on the roadway as well as assessments of how changes in the visual environment of the roadway change these probability functions. In simple terms, every object on the roadway has a probability of being seen. This probability is based on the nature of the object and the distance from which the object is seen. Logically, an object that is large and close to the driver is more easily seen than an object that is smaller and farther away. Similarly, an object that has a high visual contrast from the background, whether in color or brightness, will be more easily seen than an object with low contrast. As a driver approaches an object, the cumulative probability of seeing the object will grow until the object cannot be overlooked. The nature of this cumulative probability depends on the factors that define that object, such as contrast, size, position in the roadway, and so on.

Figure 1 shows a sample of this type of probability function. The higher-contrast object has a cumulative probability that reaches a

R. B. Gibbons and R. Bhagavathula, Virginia Tech Transportation Institute, Virginia Polytechnic Institute and State University, 3500 Transportation Research Plaza, Blacksburg, VA 24060. C. J. Edwards, HumanFirst Program, 111 Church Street SE, Minneapolis, MN 55455. P. Carlson, Texas A&M Transportation Institute, Texas A&M University System, 3135 TAMU, College Station, TX 77843-3135. D. A. Owens, Department of Psychology, Franklin and Marshall College, Lancaster, PA 17603. Corresponding author: R. B. Gibbons, gibbons@vtti.vt.edu.

Transportation Research Record: Journal of the Transportation Research Board, No. 2298, Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 96–103.
DOI: 10.3141/2298-11

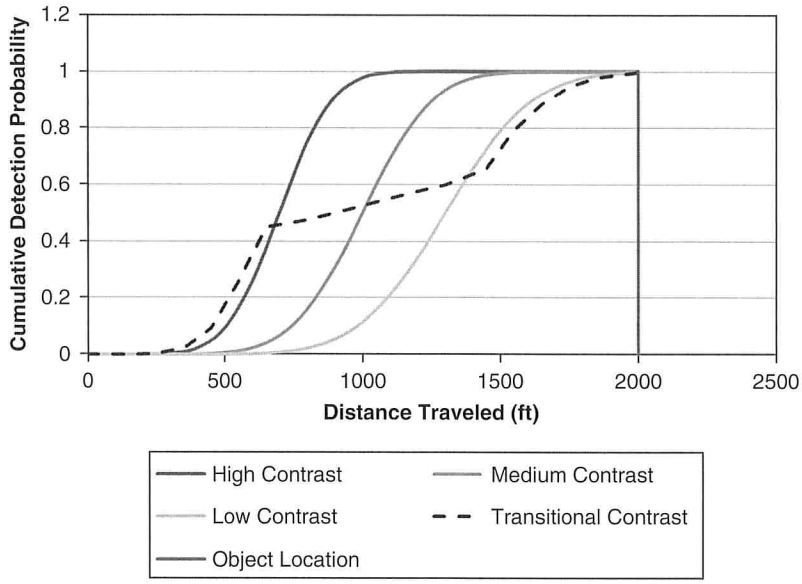


FIGURE 1 Sample cumulative probabilities of seeing an object.

theoretical level of 1 earlier than does the low-contrast object. Although this sample is obviously oversimplified, it is known that the factors that describe the object (such as contrast) change as the vehicle approaches. The transitional curve shows a potential distribution function for an object in which the contrast reverses from high contrast to low contrast as the driver approaches. In addition, not all objects achieve a cumulative probability of 1. Objects that appear off axis and in the peripheral vision may be passed before the probability of detection passes a threshold.

PURPOSE AND SCOPE

This experimental research sought to develop an initial DDVM framework, with a primary focus on identifying how the detection of different objects influences the driver visibility model. The results also provide a foundation for the visibility model and attempt to describe the nighttime visual environment.

METHODOLOGY

To define the model framework, an experimental approach was derived to manipulate and collect data on a number of different elements that influence nighttime driver behavior. Data collection was performed at the Virginia Tech Transportation Institute (VTTI) and the Texas A&M Transportation Institute (TTI). The initial factors identified for inclusion in the testing were lighting, signage, objects, glare, pavement markings, and driver age. Table 1 outlines the experimental design.

A preliminary testing effort conducted at Franklin and Marshall College provided insight into the object of interest to be studied at VTTI and TTI (*6*). The intent of having separate data collection sites was to allow for specific items of interest, some common to both sites and some unique, to be studied at two different locations so that the researchers could integrate comparative information into a model framework.

Participants

For the VTTI portion of the experiment, 24 legally licensed drivers were recruited from the New River Valley region of Virginia. Participants were balanced for gender and age. Two age ranges, representing younger and older driver populations (18 to 34 years of age and 65 years of age and older, respectively), were targeted during recruitment. TTI had a total of 31 participants, with 17 younger drivers and 12 older drivers. The younger group had a higher proportion of male drivers (i.e., 11 men) than female drivers (six women).

Equipment

Each facility used its own instrumented vehicles with separate data acquisition systems in each of the vehicles. These systems collected data on a number of variables, including kinematic data from the vehicle's internal computer system network, global positioning coordinates, steering pitch, four separate video images (driver's face, forward roadway, left side of roadway, and right side of roadway), and manual pressing of a button and other input from the in-vehicle experimenter.

TABLE 1 Data Collection Experimental Design

Factor	Level	Test Location
Age	Young, older	VTTI and TTI
Overhead lighting	No lighting, high-pressure sodium	VTTI
Signage	Two different types at varying locations	TTI
Object	Two varying types at differing locations: pedestrians, roadway obstacles	VTTI and TTI
Glare	Other vehicles present, other vehicles absent	VTTI
Pavement markings	Two different types	TTI

Headlamp Configurations

The experimental vehicles at each test facility were SUV-style vehicles with original equipment manufacturer headlamps. In each case, the headlamps were aimed according to the requirements of Federal Motor Vehicle Safety Standards (FMVSS) Regulation 108.

Lighting Test Bed of Virginia Smart Road

The Virginia Smart Road is a 2.2-mi, two-lane, controlled-access road with and without lighting. The vehicle lane width is 12 ft with a 14-ft shoulder. The Smart Road has a 0.75-mi variable-lighting system installed on the test bed. During the course of this experiment, the road was lit with 400-W high-pressure sodium luminaires in a 15-m-high, 80-m-spacing, one-sided configuration. This type of luminaire was selected from the existing luminaires on the Smart Road.

TTI Test Track

The driving course was set up as a two-lane highway with 12-ft-wide lanes located on the College Station test track. The evaluation of pavement markings included vehicle lateral position throughout the curve, pavement-marking encroachment, vehicle speed, and eye tracking. The study curves included a 200-ft, 90° deflection angle curve and a 500-ft, 45° deflection angle curve. Both the inside and outside of the curve were studied. Two levels of luminance were provided by the vehicle's low-beam and high-beam headlamps. The pavement markings were installed with waterborne paint and Type II beads. They had 30-m retroreflectivity readings of approximately 300 mcd/lx for the white markings and 240 mcd/lx for the yellow markings. The researchers used speed limit signs displaying different speeds with 10-in.-tall numbers. The speed limit signs were changed from 30 miles per hour (mph) to 55 mph throughout the course of the study. While the speed limits were changed, the participants were asked to drive at approximately 35 mph throughout the study course, ignoring the speed limit, and they were told that they might need to slow down for some of the horizontal curves. Two levels of sign sheeting materials were used (ASTM Type III and ASTM Type XI), which, when combined with two headlamp levels (i.e., low and high beams), provided up to four levels of luminance.

Collection of Eye-Tracking Data

The eye tracker used for this study was the ViewPoint EyeTracker, with EyeFrame hardware, by Arrington Research. The ViewPoint EyeTracker consists of both hardware and software components to capture and analyze eye movements. The eye-tracking hardware components are mounted to lightweight goggles that fit a variety of head sizes. Attached to the goggles are two small eye-tracking cameras and two small infrared LED illuminators that point toward each eye. There is one camera and LED illuminator per eye, and these can be adjusted to capture images of the pupil. A third camera is mounted in the center above the eyes and captures the forward scene on video. For the purposes of this study, the scene camera captured objects within the driver's forward field of view, including the vehicle interior and forward roadway. The goggles do not restrict head movement and allow participants to view any part of the scene. All of the cameras were connected to a personal computer through a frame grabber card.

Luminance Image Capture

To capture the luminance levels of the target objects used in the current study, a dynamic luminance camera setup with Point Grey cameras was used. The data transfer rate was 800 megabits/s. The cameras are capable of capturing 16-bit video and are calibrated on the basis of shutter and gain values when compared with a still photometer (*7*). The cameras were affixed to the inside of the vehicle windshield and directed toward the forward roadway.

Objects

Some of the objects used in the testing protocols were unique to one or the other of the facilities and some were used by both facilities. At the Virginia Smart Road, the test objects included pedestrians, bicyclists, small-visibility targets (e.g., gray wooden targets), deer-hunting decoys, and a white bucket launched from a compressed-air catapult by an experimenter. Some conditions called for the presence of a second vehicle, driven by the lead on-road experimenter. In some conditions in which a pedestrian was crossing the roadway, a stationary vehicle with its low-beam headlamps on was situated 80 ft in front of the pedestrian in the opposing travel lane. This glare vehicle was a sedan with original equipment manufacturer headlamps aimed according to FMVSS Regulation 108.

At the TTI facility, the objects used included a small gray wooden plaque, a pedestrian in blue medical scrubs, and the rear of a parked car. The plaque and the pedestrian were selected to replicate the work done by VTTI so that the two research teams could compare and contrast the results from the different test tracks. The rear of a parked car was included to evaluate another potential driving scenario of concern to nighttime drivers. The other objects of interest included speed signs (described in the legibility task) and pavement markings.

Procedures at Each Facility

Participants at each facility were initially screened over the telephone to identify any anomalies before the participants arrived at either test facility. Eligible participants were scheduled to come to the facility (VTTI or TTI) for one session lasting up to 3 h.

An experimenter reviewed an informed consent form with the participant upon arrival. The participant was also asked to show his or her valid driver's license. The participant completed a predrive questionnaire, which collected demographic and health information, and rated his or her comfort with nighttime driving.

The participant performed a basic visual acuity test. The vision test used a standard Snellen eye chart. The participant was asked to read the letters on the lowest line that she or he could see clearly. Participants had to have at least 20/40 vision (with or without corrective lenses) to continue with the study. Those that did not meet this criterion were paid for the amount of time they had participated in the study and were excluded from the remaining portions of the session.

VTTI Procedure

In addition to completing the general procedures already described participants at VTTI were asked to complete three more vision tests. In the second vision test, participants were asked to read a contrast sensitivity chart. This test involved indicating the direction of lines

with varying degrees of contrast and spatial frequency. In the next test, the participants were asked to read the Snellen eye chart with one eye at a time while looking through a brightness acuity tester (BAT). The BAT presented glare to the participant by using a small light directed at the eye. The participants were asked to read the lowest line they could see on the chart while looking through the BAT, which provides glare at low, medium, and high levels. The final test was for color vision, in which the experimenter asked participants to identify a series of colored numbers and different-colored backgrounds. The results of the tests after the initial visual acuity screening were for data collection purposes only and did not disqualify a participant from the study. Although not reported here, the results of these visual tests were used to identify possible outliers and data issues.

Once the participant had completed the paperwork and testing, the experimenter escorted her or him to the experimental vehicle parked in the VTTI parking lot. The in-vehicle experimenter oriented the driver to the basic vehicle controls and ensured that the participant's seat belt was fastened. The experimenter entered a rear passenger seat and prepared the data collection equipment. Once the equipment was ready, the experimenter instructed the driver to put on the eye-tracking goggles. The experimenter adjusted the eye tracker's cameras, lights, and headband such that the apparatus remained comfortable for the driver but held steadily and also accurately tracked the driver's eye movements. The experimenter then calibrated the eye tracker by asking the participant to fixate her or his eyes at certain points on a white board mounted to a vehicle in front of the participant vehicle. The experimenter used a laser pointer to indicate which points to fixate on.

Once the eye tracker was calibrated, the participant was instructed to drive to the Smart Road. The experiment began upon participant arrival on this road, and drivers were instructed to drive no faster than 40 mph. To obtain data about eye movements in unfamiliar scenarios, the study presented participants with pedestrians and other objects throughout the total of 12 laps that made up the experiment. Before the experiment, the participant received an extensive list of objects that might or might not be encountered on the Smart Road.

Objects were situated at eight stations along the Virginia Smart Road. Confederate experimenters were present at every station and could pose as pedestrians. All on-road confederates wore matching denim scrubs.

TTI Procedure

The TTI procedure had a similar goal to the procedure used at VTTI. Participants were scheduled to drive through a closed-course route at the Texas A&M University Riverside Campus at night. After consent had been granted by the participant, a researcher escorted the participant to the instrumented vehicle. Once in the vehicle, each participant was given an opportunity to become familiarized with the controls of the vehicle and adjust the vehicle seat to his or her preferences. When the participants were comfortable in the vehicle, they were directed to the eye-tracker calibration area. Here, a calibration procedure similar to the one performed at VTTI was undertaken, and participants were instructed to alert the researcher during the experiment when they recognized an object of interest.

The in-vehicle researcher guided each participant throughout the driving course. For the majority of the data collection, the researcher remained silent and allowed the participant to follow the direction of the pavement markings. Red retroreflective raised pavement markers were also placed throughout the course at key turning points and stop locations. After the final lap was completed, the participant was directed back to the office, paid for his or her time, and escorted off the premises.

COMBINED DATA ANALYSIS RESULTS

For the analysis of the combined data of VTTI and TTI, common objects and common measurement factors were entered into the logistic regression model simultaneously. Both categorical and continuous variables were entered into the model for each selected detection distance. The results and initial interpretations are presented for each of the analyses.

The variables entered into the combined logistic regression model included lighting, object type, age, glance time, number of glances, and luminance. Not all of these variables provided significant odds ratios (ORs). Some variables were significant at different distances, a finding that may require additional investigation beyond these preliminary logistical regression results. Furthermore, data were combined for some of the variables that both facilities had in common (e.g., pedestrians). Other variables were tested exclusively at one facility or the other but were still entered into the model to identify the odds of detection compared with other common objects.

Odds of Object Detection at 200 ft

The initial detection distance benchmark chosen was 200 ft. Logistic regression models were also tried at 100 ft and 150 ft. However, the logistic regression models at these distances failed to converge, which suggested a separation in the data and a failure to obtain a maximum likelihood estimate.

The logistic regression model converged at a distance of 200 ft and produced several significant ORs based on different independent variables entered into the model. Interpreting ORs can be difficult because the OR is often confused with relative risk. For example, for the detection distance of the older age group for the entire sample, the estimate result was negative, indicating that as age increased, the probability that the participant would detect an object decreased. The OR in this instance for older drivers was less than 1.0 (i.e., 0.41) (Table 2). Depending on the case, when the OR is less than 1.0, it is often easier to interpret the reciprocal (*8*).

In this instance the age-group detection was less than 1 and the reciprocal resulted in 1/0.41, yielding a value of 2.43. The results suggest that the probability of detecting an object is 2.43 times greater for younger drivers than it is for older drivers. Even at a relatively close distance to the target object, younger drivers had greater odds of detecting the target than older drivers did.

Additional significant ORs included lighting, which in this case provided a benefit in the detection of target objects. At 200 ft, the odds of detecting a target that was lit were 8.74 times greater than those of detecting an object that was not lit. At 200 ft, roadway lighting provided a clear benefit to lighting the target object as compared with the nonlighting condition, on the basis of the combination of targets, which included different-sized objects. Luminance of the target object also played a positive role, suggesting that an increase in 1 unit, or candela per square meter, increased the odds of detection by 1.2 times, thus increasing the detection of the target object, which in this case was the combination of objects.

TABLE 2 ORs and 95% Confidence Limits for Correct Detection for Certain Variables at 200 ft

Variable	Group	OR[a]	95% Confidence Limits	Significance
Age (young, old)	Old	**0.41**	−1.50, −0.29	<.01[b]
Lighting (lit, unlit)	Lit	**8.74**	1.43, 2.90	<.01[b]
Object type	Bicyclist	8.32	−1.29, 5.52	.22
	Car	**4.70**	0.63, 2.47	<.01[b]
	Deer	0.97	−0.83, 0.77	.93
	Moving pedestrian	**2.12**	−0.05, 1.55	.07[c]
	Moving pedestrian glare	1.95	−1.71, 3.05	.58
	Sign: Diamond Grade 3	**0.03**	−6.07, −0.75	.01[b]
	Sign: high-intensity beads	2.35	−0.40, 2.11	.18
	Static pedestrian	**2.52**	0.38, 1.48	<.01[b]
Glance time (s)		0.99	−0.029, 0.003	.13
Target luminance (cd/m^2)		**1.20**	0.11, 0.25	<.01[b]

[a]Bold type indicates significant and marginally significant ORs.
[b]Significant OR.
[c]Marginally significant OR.

To understand the relationship of specific objects of different sizes and different types, the analysis considered all targets common to both data collection sites and also the targets that were unique to each site (where possible) in an effort to compare across all target types. To compare and derive ORs for detecting a type of target, each target needed a reference category for the comparison. On the basis of the commonality between the two facilities and future comparison of each of the testing facilities, the gray, wooden, small-visibility target was chosen as the reference target for the object category. There are obvious size issues in the selection and comparison of target objects. However, establishing a common reference target allows comparisons across the different target types. The outcome expected from the comparisons is that the odds of detecting larger targets will be greater than those of detecting the small, wooden, gray target. Table 2 shows that this was true for some objects. For example, large objects such as the car had 4.70 times the odds of detection when compared with the wooden target. This finding likely reflects the object's size and relative motion, as well as the participants' orientation of attention toward this object of interest.

Other objects with significant and marginally significant ORs that were greater than 1 include the moving and static pedestrians. Again, the data were combined and compared between the two data collection sites. The odds of detecting a moving pedestrian were 2.12 times greater than they were for the wooden target. However, this was only marginally significant. The likely benefit here was the motion of the pedestrian and the potential hazard interest level of the target object. A similar result for the static pedestrian also occurred, with the odds of detecting a static pedestrian 2.52 times greater than those of detecting the wooden target at 200 ft. Larger targets, such as the deer decoy and the bicyclist, did not have significant ORs associated with them. The obvious size difference between these targets and the comparison target would suggest that the odds of detecting larger targets would be significantly greater. However, for these specific targets, that was not the case. These results suggested that, at this range, detection of these targets did not differ from detection of the small wooden target. There was no additional advantage for these target types based on size.

The final significant OR at 200 ft was that of the sign with diamond-grade sheeting. Interestingly, it had a significant OR of less than 1 and, as with the age-group calculation, the interpretation is easier if the reciprocal is applied. On the basis of the reciprocal, the odds of detecting the small, wooden target (again the comparison object for this group) were 33.3 times greater than those of detecting the diamond-grade sign at 200 ft. These results may reflect the interest in the sign at this distance, which may suggest that once the sign has been detected (at a greater distance) and the sign information has been conveyed to the driver, the visual search is then directed elsewhere even when the driver is approaching the sign.

Odds of Object Detection at 500 ft

As the detection distance cutoff increased, the odds of target detection were also manipulated on the basis of the independent variables of interest (Table 3). Older drivers were still at a disadvantage in detection, with younger drivers having 2.63 (i.e., 1/0.38) times greater odds of detection when compared with older drivers.

TABLE 3 ORs and 95% Confidence Limits for Correct Detection for Certain Variables at 500 ft

Variable	Group	OR[a]	95% Confidence Limits	Significance
Age (young, old)	Old	**0.38**	−1.42, −0.51	<.001
Lighting (lit, unlit)	Lit	**2.98**	0.48, 1.70	.01
Object type	Bicyclist	**10.02**	1.49, 3.12	<.01
	Car	**64.38**	2.92, 5.41	<.01
	Moving pedestrian	**6.89**	1.26, 2.60	<.01
	Moving pedestrian glare	**2.71**	0.17, 1.82	.02
Target luminance (cd/m^2)		**1.28**	0.02, 0.48	.03

[a]Bold type indicates significant ORs.

There were several significant ORs for object type at this distance. For example, the odds of detection in comparison with the wooden target increased for the bicyclist (OR = 10.02), car (OR = 64.38), moving pedestrian (OR = 6.89), and moving pedestrian glare (OR = 2.71). The high OR for the car suggests not only that size played a substantial role in the increased odds of detection, but also that the vehicle might be a target of substantial interest when compared with the other target objects.

Last, lighting had a significant OR at the 500-ft detection cutoff. Correct object detection was 2.98 times greater for lighted conditions than for nonlighted conditions. Target luminance also aided detection, with a 28% greater detection rate for each unit increase in target luminance.

One difference encountered at this cutoff distance was the nonsignificant effect of signage. This may suggest that the signs were still identified before this point and produced a significant difference in signage detection compared with the reference target.

Odds of Object Detection at 750 ft

The final combined cutoff distance was set at 750 ft. Logistic regression models were tested on the basis of longer detection distances (Table 4). However, as a result of a complete separation of data, none of the models converged beyond the 750-ft distance.

Similar to previous cutoff distances, younger drivers had 2.12 greater odds of object detection compared with older drivers. At this distance from the target object, older drivers were at a disadvantage compared with their younger counterparts.

Target objects all had substantially greater odds of detection compared with the reference target. The significant ORs for the bicyclist (OR = 60.45), car (OR = >1,000), moving pedestrians (OR = 33.83), and moving pedestrian glare (OR = 14.03) showed these items had greater odds of detection when compared with the gray, wooden target. Interestingly, both signs at 750 ft showed significant ORs. The diamond-grade sign had 739.45 times greater detection odds than the reference target, and the high-intensity bead sign had 115.26 times greater detection rates. These results suggest that the signage was observed at high detection distances. At this distance, the diamond-grade sign had a greater OR than the high-intensity bead signs.

The last results were for target luminance. At 750 ft, the significant OR was less than 1. Thus, for interpretation purposes, the reciprocal was used (1/0.89). The results suggest that with every unit increase in target luminance, object detection decreased by 12%. This is an interesting result that may point to a change in the relationship of the target luminance and the luminance contrast of the object. At short distances the target luminance may be highly correlated with contrast, but this may not be the case at long distances. This is a point for which further research may be required.

DISCUSSION OF RESULTS

Reviewing the data from all of the distances shows interesting trends for the impact of the object type and the luminance on the probability of detection and the application of the DDVM (see Figure 2). The younger age group continually outperformed the older counterparts at every cutoff distance within the model. This finding suggests that the older drivers had difficulty detecting objects and that a combined DDVM must consider the aging eye as a factor when visibility is modeled.

Roadway lighting provided consistent increased odds of detection across the different cutoff distances. Even as distance increased, lighting still provided a positive benefit for detection objects. The continual positive ORs also reflect the importance of this variable with respect to the DDVM. This result, however, may not be evident for all roadway lighting but just for the case tested in this study. Ambient conditions, geometry, and other factors, such as traffic volume, may influence this impact.

The ORs for target luminance were significant throughout the different cutoff values. The combined data revealed positive influences on target detection on the basis of target luminance. The odds of target detection increased 14% to 29% for every unit increase in the target luminance. Interestingly, however, this effect was only in evidence for detection distances between 200 and 500 ft. A nonsignificant OR was obtained for the 600-ft distance, and a significant decline was obtained for the 750-ft detection distance. At 750 ft, detection distance decreased by 12% for every unit increase in the predictor, in this case target luminance (in candelas per square meter). The reversal at longer detection distances, much like those experienced in the TTI data by itself, may reflect the impact of contrast reversal at longer detection distances. Overall, the combined data from the VTTI and TTI data collection effort provided valuable information on the variables of interest for the DDVM.

CONCLUSIONS

The entire research group provided insight and developed several research capabilities in an effort to measure visual information from a driver's perspective. This project considered three aspects of the

TABLE 4 ORs and 95% Confidence Limits for Correct Detection for Certain Variables at 750 ft

Variable	Group	OR[a]	95% Confidence Limits	Significance
Age (young, old)	Old	**0.47**	−1.35, −0.17	.01
Object type	Bicyclist	**60.45**	2.04, 6.16	<.0001
	Car	**>1,000.00**	6.86, 11.33	<.0001
	Moving pedestrian	**33.83**	1.56, 5.48	.00
	Moving pedestrian glare	**14.03**	0.54, 4.74	.01
	Sign: Diamond Grade 3	**739.45**	3.48, 9.73	<.0001
	Sign: high intensity beads	**115.26**	2.40, 7.10	<.0001
Target luminance (cd/m^2)		**0.89**	−0.18, −0.07	<.0001

[a]Bold type indicates significant ORs.

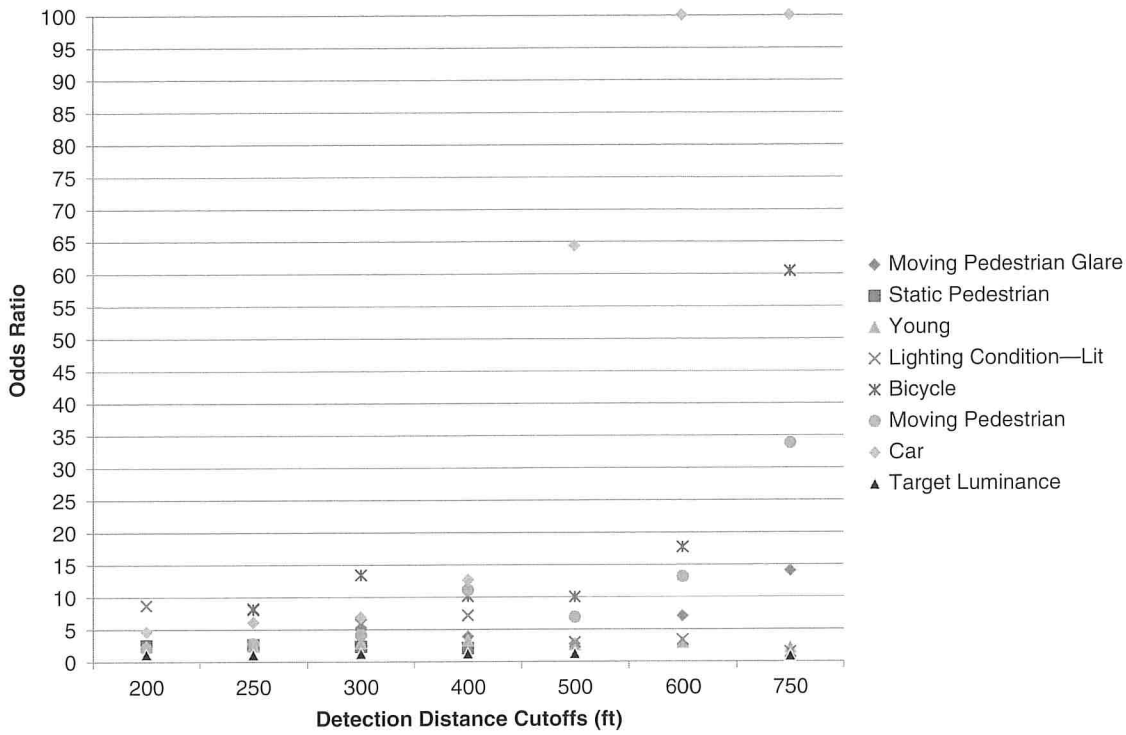


FIGURE 2 Combined ORs and detection distances.

creation of the model framework: a technological solution for driver monitoring, a framework for the visual model to be developed, and data collection and analysis methodology.

The technological solution required the combination of state-of-the-art photometric measurements and eye-tracking technologies into a single driver visual monitoring system. The system assessed the visual activity of a driver as well as the lighting characteristics of the scene and then linked the data for reduction purposes.

The creation of the DDVM framework required identification of the specific nature of the model and how it would be used to develop and assess new roadway technologies. The model needed flexibility to assess the impact of roadway technologies and applications on a driver's visual behavior. The model framework review provided insight into the necessary requirements or elements for the modeling effort. The framework was based on the current knowledge of human visual functionality, performance, and other characteristics.

Data analysis and collection methods were also reviewed and developed. A series of experimental sessions was defined, these sessions included a variety of visual elements in a nighttime driving environment. Data collected from these research efforts were then submitted to a modeling logistical regression analysis to assess overall fit. The information obtained from this analysis provided insight into a potential model that could incorporate information from a number of variables and predict, with some reasonable accuracy, the chance of driver detection.

The following list provides an overview of the data collected and the modeling framework for visibility:

- Moving objects, especially those in the lit condition, were detected sooner than those in the unlit condition. Lighting that makes pedestrian and bicyclist movement visible provides a large attentional draw for a driver.
- As expected, larger objects were detected sooner than smaller objects. However, these results can be modified by type of material (e.g., signage).
- Age effects, as expected, influenced detection distance of objects at the various cutoff distances.
- Target luminance influenced detection of target objects, especially at longer distances.
- The logistic regression technique provided a useful tool for the initial model framework. Additional data collection and variable interactions will provide increased model reliability.

Limitations

This research effort had several limitations that provide insight into future iterations of a visual modeling effort. Although eye glance and target luminance data were entered into the overall model, they were found not to affect the odds of object detection as highly as anticipated. This finding may have been a result of the potential to have significant data losses in the collection of eye-tracking and luminance data. This loss affected the utility of both the eye movement data and the luminance data collected during each study session.

Finally, interactions between the variables were not directly addressed in the current analysis effort. These interactions are often very difficult to interpret in the light of a logistical regression model. Further statistical insight and model refinement is required for additional accuracy and predictive capabilities.

Future Considerations and Improvements

The current research effort set out to identify and investigate the applicability of a potential model and the functionalities required

to develop the model. The results presented suggest the need for further investigation and refinement of the model framework, statistical analysis, and also additional variables that describe the roadway environment. Additional elements should include the following:

- Development of the initial driver model. The factors explored in the initial model framework should be investigated in greater detail, allowing for a base model of driver visual behavior to be developed.
- Additional factors. The current research describes some initial factors for driver nighttime visual behavior. Additional factors (e.g., geometry) should be taken into account for more robust and complete modeling information.
- Validation and real-world testing. To qualify the model, real-world testing should be established such that validation can occur.

ACKNOWLEDGMENTS

The research reported here was performed under FHWA's Exploratory Advanced Research Program by the Virginia Tech Transportation Institute with the cooperation of the Texas A&M Transportation Institute and Franklin and Marshall College. The authors thank Carl Andersen for his assistance and oversight during the research effort.

REFERENCES

1. Ising, K. W., T. R. Fricker, J. M. Lawrence, and G. P. Siegmund. *Threshold Visibility Levels for the Adrian Visibility Model Under Nighttime Driving Conditions.* Technical Paper 2003-01-0294. Society of Automotive Engineers, Warrendale, Pa., 2003.
2. Pretto, P., and A. Chatziastros. The Role of Scene Contrast and Optic Flow on Driving Speed. *Proc., 11th International Conference on Vision in Vehicles.* North-Holland, Amsterdam, Netherlands, 2007, pp. 1–8.
3. Paulmier, G., C. Brusque, V. Carta, and V. Nguyen. The Influence of Visual Complexity on the Detection of Targets Investigated by Computer Generated Images. *Lighting Research and Technology,* Vol. 33, No. 3, 2001, pp. 197–207.
4. Adrian, W. Visibility of Targets: Model for Calculation. *Lighting Research and Technology,* Vol. 21, No. 4, 1989, pp. 180–188.
5. Rea, M. S., and M. J. Ouellette. Relative Visual Performance: A Basis for Application. *Lighting Research and Technology,* Vol. 1, No. 3, 1991, pp. 135–144.
6. Owens, D. A., M. Hunter, X. Zhang, R. B. Gibbons, C. J. Edwards, and D. Viita. Assessment of Nighttime Visibility of Realistic Targets Using a Video-Based Simulation of Virginia Smart Road. Presented at 89th Annual Meeting of the Transportation Research Board, Washington, D.C., 2010.
7. Meyer, J. E., R. B. Gibbons, and C. J. Edwards. *Development and Validation of a Luminance Camera.* Report 09-UL-003. National Surface Transportation Safety Center of Excellence. Virginia Tech Transportation Institute, Blacksburg, 2009. http://scholar.lib.vt.edu/VTTI/reports/Luminance_Camera_021109.pdf
8. Pedhazur, E. J. *Multiple Regression in Behavioral Research: Explanation and Prediction,* 3rd ed. Harcourt Brace, New York, 1997.

The Visibility Committee peer-reviewed this paper.

TRANSPORTATION RESEARCH RECORD:
JOURNAL OF THE TRANSPORTATION RESEARCH BOARD

Peer Review Process

The ***Transportation Research Record: Journal of the Transportation Research Board*** publishes approximately 25% of the more than 4,000 papers that are peer reviewed each year. The mission of the Transportation Research Board (TRB) is to disseminate research results to the transportation community. The Record series contains applied and theoretical research results as well as papers on research implementation.

The TRB peer review process for the publication of papers allows a minimum of 30 days for initial review and 60 days for rereview, if needed, to ensure that only the highest-quality papers are published. A minimum of three reviews are required for a publication recommendation. The process also allows for scholarly discussion of any paper scheduled for publication, along with an author-prepared closure.

The basic elements of the rigorous peer review of papers submitted to TRB for publication are described below.

Paper Submittal: June 1–August 1

Papers may be submitted to TRB at any time. However, most authors use the TRB web-based electronic submission process available between June 1 and August 1, for publication in the following year's Record series.

Initial Review: August 15–November 15

TRB staff assigns each paper by technical content to a committee that administers the peer review. The committee chair assigns at least three knowledgeable reviewers to each paper. The initial review is completed by mid-September.

By October 1, committee chairs make a preliminary recommendation, placing each paper in one of the following categories:

1. Publish as submitted or with minor revisions;
2. Reconsider for publication, pending author changes and rereview; or
3. Reject for publication.

By late October, TRB communicates the results of the initial review to the corresponding author. Corresponding authors communicate the information to coauthors. Authors of papers in Category 2 (above) must submit a revised version addressing all reviewer comments, along with an explanation of how the comments have been addressed.

Rereview: November 20–January 25

The committee chair reviews revised papers in Category 1 (above) to ensure that the changes are made and sends the Category 2 revised papers to the initial reviewers for rereview. After rereview, the chair makes the final recommendation on papers in Categories 1 and 2. If the paper has been revised to the committee's satisfaction and ranks among the best papers, the chair may recommend publication. The chair communicates the results of the rereview to the authors.

Discussions and Closures: February 1–May 15

Discussions may be submitted for papers that will be published. TRB policy is to publish the paper, the discussion, and the author's closure in the same Record.

Many papers considered for publication in the *Transportation Research Record* are also considered for presentation at TRB meetings. Individuals interested in submitting a discussion of any paper presented at a TRB meeting must notify TRB no later than February 1. If the paper has been recommended for publication in the *Transportation Research Record*, the discussion must be submitted to TRB no later than April 15. A copy of this communication is sent to the author and the committee chair.

The committee chair reviews the discussion for appropriateness and asks the author to prepare a closure to be submitted to TRB by May 15. The committee chair reviews the closure for appropriateness. After the committee chair approves both discussion and closure, the paper, the discussion, and the closure are included for publication together in the same Record.

Final Manuscript Submittal: March 15

In early February, TRB requests a final manuscript for publication—to be submitted by March 15—or informs the author that the paper has not been accepted for publication. All accepted papers are published by December 31.

Paper Awards: April to January

The TRB Executive Committee has authorized annual awards sponsored by Groups in the Technical Activities Division for outstanding published papers:

- Charley V. Wootan Award (Policy and Organization Group);
- Pyke Johnson Award (Planning and Environment Group);
- K. B. Woods Award (Design and Construction Group);
- D. Grant Mickle Award (Operations and Preservation Group);
- John C. Vance Award (Legal Resources Group);
- Patricia F. Waller Award (Safety and System Users Group); and
- William W. Millar Award (Public Transportation Group).

Other Groups also may nominate published papers for any of the awards above. In addition, each Group may present a Fred Burggraf Award to authors 35 years of age or younger.

Peer reviewers are asked to identify papers worthy of award consideration. Each Group reviews all papers nominated for awards and makes a recommendation to TRB by September 1. TRB notifies winners of the awards, which are presented at the following TRB Annual Meeting.

Transportation Research Board
www.TRB.org

TRANSPORTATION RESEARCH BOARD
500 Fifth Street, NW
Washington, DC 20001

ADDRESS SERVICE REQUESTED

NON-PROFIT ORG.
U.S. POSTAGE
PAID
WASHINGTON, D.C.
PERMIT NO. 8970

THE NATIONAL ACADEMIES™

Advisers to the Nation on Science, Engineering, and Medicine

The nation turns to the National Academies—National Academy of Sciences, National Academy of Engineering, Institute of Medicine, and National Research Council—for independent, objective advice on issues that affect people's lives worldwide.

www.national-academies.org

ISBN 978-0-309-22334-8
90000
9 780309 223348